THE ARCHITECTS' AND BUILDERS' HANDBOOK

THE ARCHITECTS' AND BUILDERS' HANDBOOK

DATA FOR
ARCHITECTS, STRUCTURAL ENGINEERS,
CONTRACTORS, AND DRAUGHTSMEN

BY

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SEVENTEENTH EDITION, ENLARGED
TOTAL ISSUE, ONE-HUNDRED AND FIVE THOUSAND

NEW YORK
JOHN WILEY & SONS, Inc.
LONDON: CHAPMAN & HALL, LIMITED

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early editions of books are free from errors; but it is the endeavor of t
Publishers to have them removed, and it is therefore desired that the Edit
in-Chief may be aided in his task of revision, from time to time, by the kinc
criticism of readers.

JOHN WILEY & SONS, INC.
432 FOURTH AVENUE, NEW YO

Printed in U. S. A.

7/26 COMPOSITION, ELECTROTYPING, PRINTING AND BINDING BY
BRAUNWORTH & CO., BROOKLYN, N. Y.

This Book

IS RESPECTFULLY DEDICATED TO THOSE WHOSE KINDNESS HAS ENABLED ME TO PRODUCE IT

TO MY PARENTS

WHO GAVE ME THE EDUCATION UPON WHICH IT IS BASED.

TO MY WIFE

FOR HER LOVING SYMPATHY, ENCOURAGEMENT AND ASSISTANCE

TO ORLANDO W. NORCROSS

OF WORCESTER, MASS.

WHOSE SUPERIOR PRACTICAL KNOWLEDGE OF ALL THAT PERTAINS TO BUILDING HAS GIVEN ME A MORE INTELLIGENT AND PRACTICAL VIEW OF THE SCIENCE OF CONSTRUCTION THAN I SHOULD OTHERWISE HAVE OBTAINED *

* Dedication to First Edition.

EDITORIAL STAFF

NOTE TO SEVENTEENTH EDITION

WITH this edition the name is changed from Pocket-Book to Handbook
The work has been revised, some chapters rewritten, new chapters addec
and a new Index made. Many cuts have been reengraved.

The twenty-nine chapters of Part II have been revised where necessary t
make the data agree with the latest research and practice, and two new chapter
have been added, Chapter XXX, on Specifications for the Steelwork of Builc
ings, by Robins Fleming, and Chapter XXXI, on Domical and Vaulted Struc
tures, by Edward F. Ries. Chapter XXIII, on Fireproofing of Buildings, an
Chapter XXIV, on Reinforced-Concrete Construction, have been rewritten b
Rudolph P. Miller. In Part III, the sections on Heating and Ventilation, an
Chimney Construction, have been entirely rewritten by Louis A. Harding
The new chapters and sections include numerous practical examples of every
day problems, with solutions worked out in complete detail.

In addition to the new chapters, numerous new articles have been added t
the text and illustrations of Part II, on the following subjects: New Data o
Building Laws Relating to Loads on Masonry; Graphical Method of Determin
ing the Center of Gravity of Plane Figures or Sections; Graphical Method o
Determining Moments of Inertia of Plane Figures; Triangular Loading; Enc
Connections of Tension-Members; New Wire-Data; New Matter on Gauges
New Matter on Chains; Graphical Method of Determining the Deflection o
Beams; Secondary Stresses; Angles Used as Beams; Data on Girderless
Reinforced-Concrete Floors; Data on Tanks, and on Stresses in Cylindrical
Tanks, etc. Other revisions and additions have been made, including new
sections relating to the Registration of Architects, Standard Documents of the
American Institute of Architects, Architectural Education, etc.

In its revised, compact, and convenient form, the Editor believes that the work, more than ever before, will maintain its preeminence as the authoritative Handbook of Building-Construction.

PHILADELPHIA, July, 1921.

PREFACE TO SIXTEENTH EDITION

The changes in the fifteenth edition, published in 1908, consisted principally of the rewriting of the two chapters on Fireproofing of Buildings and Reinforced Concrete.

In 1912 the undersigned was asked to undertake the revision of the entire book with the cooperation of a corps of Associate Editors, each highly qualified to render the necessary assistance in matters pertaining to his own work. On account of the comprehensive nature of the contents of the Pocket-Book, the many recent changes and rapid developments in different fields of architectural construction, and the consequent effect of such changes on the interrelated subjects treated, the Editor-in-Chief decided to rewrite and reset the entire book. After more than three years of arduous labor, in which the Associate Editors and many other contributors have most ably and generously assisted, the New Kidder is about to be published.

It was decided to retain Mr. Kidder's original arrangement of the subject-matter which is divided into three Parts, Part I dealing with practical applications of Arithmetic, Geometry and Trigonometry, Part II with the Materials of Construction and the Strength and Stability of Structures, and Part III with miscellaneous useful information for architects and builders. Each of the twenty-nine chapters of Part II, however, has the name of the Associate Editor who revised or rewrote it printed with the chapter-caption. Part I has been carefully checked and much of the matter rearranged. The twenty-eight chapters of Part II have been rewritten and one new chapter has been added on Reinforced-Concrete Mill and Factory-Construction. Part III has been largely rewritten and all subjects retained have been thoroughly revised. To this part, also, much new matter has been added, such as extended tables of Specific Gravities and Weights of Substances, Architectural Acoustics, Waterproofing for Foundations, the Quantity System of Estimating, the Standard Documents of the American Institute of Architects, Educational Societies of the World and extended lists of Architectural Schools, Books and Periodicals.

The Editor-in-Chief has, with very few exceptions, personally checked on every page of manuscript, galley-proof and page-proof the equations, formulas, computations and problems, and has read or examined carefully every word, figure and illustration, every detail of syntax, paragraphing, punctuation and typography, and every arrangement of tables, captions, classifications, notation, Table of Contents and Index.

He is responsible for many changes in the form of presentation of data which it is hoped will add to the Pocket-Book still more of that efficiency and practical usefulness for which it has been so long noted. Some of these changes may be briefly mentioned. The text has been entirely reset; the type, while slightly smaller, is clearer and has the lines and paragraphs separated by wide leads; a special type is used for the tables; the paragraphing is revised throughout and every paragraph has a black-face type caption descriptive of the subject-matter; words in italics or with quotation-marks are seldom used, words in small caps taking their place; every chapter is divided into numbered chapter-subdivisions which are briefly descriptive of the classified matter; the number of cross-

references is largely increased and the page-numbers of such references are alm always added; many tables and diagrams which in the former editions r lengthwise of the page have been reset or reengraved to read across the page greater convenience; the number of illustrations has been largely increased, ma old cuts reused have been reengraved, and some diagrams printed with lines different colors to make the demonstrations clearer; a descriptive caption l been added to every illustration; the abbreviations Chap. I, Chap. II, etc., ha been printed with each page-caption of the left-hand pages, thus avoiding t necessity of referring to the Table of Contents to locate any particular chapt

The Editor-in-Chief decided to change some of the unit stresses, especia those for the different woods, and in some cases to recommend more conservat values, and he believes that results based upon such stresses conform to the b engineering practice. This change necessitated the revision of many tables a problems throughout the book which had to be entirely recalculated. Num ous practical problems with complete solutions have been added. The deri tion of many of the formulas used has been explained, either in the body of t text or in extended foot-notes, for those who wish to understand as well as to u such formulas, and numerous cross-references accompanying them enable t reader to use the Pocket-Book as a textbook for certain parts of the mechan of materials as well as a handbook for office work. The tables of the propert of structural shapes, of safe loads for columns, beams and girders, etc., have be revised and numerous new tables added. The Editor has found that it is the co sensus of opinion among architects that the insertion of these tables is a gre convenience and for their ordinary office work condenses into one handy v ume much of the essential data of several manufacturers' handbooks.

The difficulty of securing a unity of treatment and of avoiding repetitions a contradictions in a book of reference the data of which covers so many subje and is written by so many contributors has been fully realized; but it is believ that in these respects the New Kidder is reasonably successful and will me with the approval of all who use it.

Acknowledgments and thanks are due the Associate Editors for their hear cooperation and generous contributions of the time and labor taken from th professional work. Acknowledgment is made, also, of the valuable assistan of all others who have furnished new or revised old data, and of many helpf suggestions from Mrs. F. E. Kidder and from the publishers.

The Editor-in-Chief expresses the hope that for the architects and builde of this country the new Pocket-Book will continue to be, as Mr. Kidder express it in his preface to the first edition in 1884, "a compendium of practical fact rules and tables presented in a form as convenient for application as possibl and as reliable as our present knowledge will permit;" and also, in its prese extension and fuller development, a work which will lead to a still clearer unde standing of the essential principles of sound architectural construction.

THOMAS NOLAN.

PHILADELPHIA, September, 1915.

PREFACE TO FOURTEENTH EDITION

It is now nearly twenty years since the author, then quite a young man, completed the first edition of this work, which, although containing but 586 pages, had required about three years for its preparation. At that time the author thought he had covered all of those practical details relating to the planning and construction of buildings, with which the architect was concerned, reasonably well, and it would appear as though the purchasers of the book thought so too, but as the years have come and gone, so many and such great improvements have taken place in the building world, so many articles invented, new methods of construction developed, higher standards established, that the present edition, although containing nearly three times as many pages, is perhaps not more complete, for the times, than was the first edition.

When preparing the first edition, it was the aim of the author to give to architects and builders a handbook which should be, in its field, as useful and reliable as Trautwine's had been to civil engineers; and with that object constantly in view, the book has been revised from time to time to meet the changed conditions in building construction and equipment.

About three years ago it was thought, by the publishers and the author, that a thorough and complete revision of the book should be undertaken, and although the re-writing of a work of this character, even with the thirteenth edition to work from, involved many months of close and constant application, the utilization of those hours which one ordinarily takes for recreation, and at the best more or less interruption to his regular business, and consequent reduction in income, the writer undertook to prepare a work of a still wider scope, and which should be thoroughly up-to-date in every particular, or at least as far as is practicable, in a work requiring a period of three years in its preparation, and from that time to this he has spared no labor or expense to make the book as useful and complete as he possibly could, without making it too bulky.

In this revision the author has had in view:

1st. A reference-book which should contain some information on every subject (except design) likely to come before an architect, structural engineer, draughtsman or master-builder, including data for estimating the approximate cost.

2d. To as thoroughly cover the subject of architectural engineering as is practicable in a handbook.

3d. To present all information in as simple and convenient a form for immediate application as is consistent with accuracy. To this end a great many new tables, arranged and computed by the author, have been inserted.

At the time the first edition was written, the term "Architectural Engineering" had not been used in its present application, and the term "Structural Engineering," when used, referred almost exclusively to bridge work.

To-day, structural and architectural engineers are concerned almost exclusively with building construction, and their work is more closely allied to that of the architect than to that of the civil engineer; hence the author has had in mind the needs of the structural engineer and draughtsman as well as those of the architect and builder, and the book should be of nearly equal value to both.

Where it was impossible, for lack of space, to go extensively into any sub references to other books or sources of information have been given, so tha this way the book may serve as a general index to the many lines of w materials, and manufactured products entering into the planning, construc and equipment of buildings.

To attain the objects in view, it has been necessary to add considerably tc number of pages, but as experience has shown that the book is used princip at the desk or draughting-table, and is seldom carried in the pocket, it is beli that the convenience of having everything in one book will more than offset disadvantage resulting from increase in bulk.

Nearly the entire book has been re-written, and great pains have been ta to furnish reliable data. A large number of experts in various lines have assi the author, as is manifest by the foot-notes and references. To all of such, to the many authors of technical works, and to the publishers of techn journals, who have kindly consented to the use of cuts and data, the aut takes pleasure in acknowledging his indebtedness. Also to Mr. E. S. Hand New York, who, for many years, has rendered material assistance in collec data along the line of manufactured products.

The names and addresses of manufacturers have been given solely for convenience of the users of the book, and not for any pecuniary considerati in fact, if money considerations had solely appealed to the writer, this book wc never have been re-written, because a technical work of this character can ne adequately compensate, in money, for the time, labor, and thought require its preparation. The many words of appreciation which have come to the aut from hundreds of those who have found the book useful have been a gr stimulus to further increase its usefulness.

As in the former prefaces, the author requests that any one discovering err in the work or who may have any suggestions looking to the further impro ment of the book, will communicate the same to him, that the book may be m as complete and reliable as possible.

Finally, the author desires to acknowledge his indebtedness to the publish who have heartily seconded his efforts in every particular, and who have spa no pains or expense to make a perfect handbook.

F. E. KIDDER

DENVER, COLO., July 18th, 1904.

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PART I

PRACTICAL
RITHMETIC, GEOMETRY AND TRIGONOMETRY

RULES, TABLES AND PROBLEMS

1. PRACTICAL ARITHMETIC

Mathematical Signs and Characters*

The following signs and characters are generally used to denote and abbreviate the several mathematical operations:

The sign = means equal to, or equality;
− means minus or less, or subtraction;
+ means plus, or addition;
× means multiplied by, or multiplication;
÷ or / means divided by, or division;
² ³ are indexes or powers, meaning that the number to which they are added is to be squared (²) or cubed (³);
: is to, : : so is, : to are signs of proportion;
√ is the RADICAL SIGN and means that the square root of the number before which it is placed is to be extracted;
∛ means that the cube root of the number before which it is placed is to be extracted;
—— the BAR indicates that all the numbers under it are to be taken together;
( ) the PARENTHESIS means that all the numbers between are to be taken as one quantity;
. means decimal parts; thus, 2.5 means $2\frac{5}{10}$, 0.46 means $\frac{46}{100}$.
° means degrees, ′ minutes and ″ seconds;
∴ means hence;
′ means feet;
″ means inches.

Involution

To Square a Number, multiply the number by itself, and the product will be the square; thus, the square of 18 = $18^2 = 18 \times 18 = 324$.

The Cube of a Number is the product obtained by multiplying the number by itself, and that product by the number again; thus, the cube of 14 = $14^3 = 14 \times 14 \times 14 = 2\,744$.

The Fourth Power of a Number is the product obtained by multiplying the number by itself four times; thus, the fourth power of 10 = $10^4 = 10 \times 10 \times 10 \times 10 = 10\,000$.

Evolution

Square Root. Rule for extracting the square root of a number:

(1) Divide the given number into periods of two figures each, commencing at the right if it is a whole number, and at the decimal point if there are decimals; thus, $1\dot{0}2\dot{3}6.\dot{8}1\dot{2}6$.

(2) Find the largest square in the left-hand period, and place its root in the quotient; subtract the said square from the left-hand period, and to the remainder bring down the next period for a new dividend.

(3) Double the root already found, and annex one cipher for a trial-divisor; see how many times it will go in the dividend, and put the number in the quotient

* See, also, pages 122 and 123, Part II.

and also in place of the cipher in the divisor. Multiply this final divisor by the number in the quotient just found, subtract the product from the dividend and to the remainder bring down the next period for a new dividend and proceed as before. If it should be found that the trial divisor cannot be contained in the dividend, bring down the next period for a new dividend, annex another cipher to the trial divisor, put a cipher in the quotient and proceed as before.

Example.

```
             10236.8126 (101.17, the square root
          1
     201)0236
          201
    2021)3581
         2021
   20227)156026
         141589
          14437
```

Cube Root. To extract the cube root of a number, point off the number from right to left into periods of three figures each, and, if there is a decimal, commence at the decimal point and point off into periods, going both ways.

Ascertain the highest root of the first period, and place it to the right of the number, as in long division; cube the root thus found and subtract from the first period; to the remainder annex the next period; square the root already found, multiply by three and annex two ciphers for the trial divisor. Find how many times this trial divisor is contained in the dividend and write the result in the root.

Add together the trial divisor, three times the product of the first figure of the root by the second with one cipher annexed, and the square of the second figure in the root; multiply the sum by the last figure in the root, and subtract from the dividend; to the remainder annex the next period and proceed as before.

When the trial divisor is greater than the dividend, write a cipher in the root, annex the next period to the dividend and proceed as before.

Example. Required, the cube root of 493039 or $\sqrt[3]{493039}$

```
                        493039(79, the cube root
           7×7×7 =      343
         7×7×3 = 14700 | 150039
         7×9×3 =  1890 |
           9×9 =    81 |
                 16671 | 150039
```

Example. Required, the cube root of 403583.419 or $\sqrt[3]{403583.419}$

```
                           403583.419(73.9, the cube root
               7×7×7 =     343
             7×7×3 = 14700    | 60583
             7×3×3 =   630    |
               3×3 =     9    |
                     15339    | 46017
        73×73×3 = 1598700     | 14566419
        73× 9×3 =   19710     |
            9×9 =      81     |
                  1618491     | 14566419
```

Example. Required, the cube root of 158252.632929 or $\sqrt[3]{158252.632929}$

```
                                 158252.632929(54.09, the cube root
                5 × 5 × 5 = 125
           5 × 5 × 3 = 7500 | 33252
           5 × 4 × 3 =  600 |
               4 × 4 =   16 |
                       8116 | 32464
 540 × 540 × 3 = 87480000   |   788632929
 540 ×   9 × 3 =   145800   |
         9 × 9 =       81   |
                 87625881   |   788632929
```

TABLES

OF

SQUARES, CUBES, SQUARE ROOTS, CUBE ROOTS AND RECIPROCALS

From 1 to 1054

The following table, taken from Searle's Field Engineering, will be found of great convenience in finding the square, cube, square root, cube root and reciprocal of any number from 1 to 1054. The reciprocal of a number is the quotient obtained by dividing 1 by the number. Thus, the reciprocal of 8 is $1 \div 8 = 0.125$.

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
1	1	1	1.0000000	1 [redacted]	1.000000000
2	4	8	1.4142136	1.2599210	.500000000
3	9	27	1 [redacted]	1.4422496	.333333333
4	16	64	2.0000000	1.5874011	.250000000
5	25	125	2.2360680	1.7099759	.200000000
6	36	216	2.4494897	1.8171206	.166666667
7	49	343	2.6457513	1.9129312	.142857143
8	64	512	2.8284271	2.0000000	[redacted]
9	81	729	3 [redacted]	2.0800837	.111111111
10	100	1000	3.1622777	2 1544347	.100000000
11	121	1331	3.3166248	2.2239801	.090909091
12	144	1728	3.4641016	2.2894286	.083333333
13	169	2197	3.6055513	2.3513347	.076923077
14	196	2744	3.7416574	2.4101422	.071428571
15	225	3375	3.8729833	2.4662121	.066666667
16	256	4096	4.0000000	2.5198421	.062500000
17	289	4913	4.1231056	2.5712816	.058823529
18	324	5832	4.2426407	2.6207414	.055555556
19	361	6859	4.3588989	2.6684016	.052631579

No.			Square roots	Cube
63	3969	250047	7.9372539	3.979
64	4096	262144	8.0000000	4.000
65	4225	274625	8.0622577	4.020
66	4356	287496	8.1240384	4.041
67	4489	300763	8.1853528	4.061
68	4624	314432	8.2462113	4.081
69	4761	328509	8.3066239	4.101
70	4900	343000	8.3666003	4.121
71	5041	357911	8.4261498	4.140
72	5184	373248	8.4852814	4.160
73	5329	389017	8.5440037	4.179
74	5476	405224	8.6023253	4.198
75	5625	421875	8.6602540	4.217
76	5776	438976	8.7177979	4.235
77	5929	456533	8.7749644	4.254
78	6084	474552	8.8317609	4.272
79	6241	493039	8.8881944	4.290
[illegible]	6400	512000	8.9442719	4.308
[illegible]	6561	531441	9.0000000	4.326
[illegible]	6724	551368	9.0553851	4.344
[illegible]	6889	571787	9.1104336	4.362
[illegible]	7056	592704	9.1651514	4.379
[illegible]	7225	614125	9.2195445	4.396
[illegible]	7396	636056	9.2736185	4.414
[illegible]	7569	658503	9.3273791	4.431
[illegible]	7744	681472	9.3808315	4.447
[illegible]	7921	704969	9.4339811	4.464
90	8100	729000	9.4868330	4.481
91	8281	753571	9.5393920	4.497
92	8464	778688	9.5916630	4.514
93	8649	804357	9.6436508	4.530
94	8836	830584	9.6953597	4.546
95	9025	857375	9.7467943	4.562
96	9216	884736	9.7979590	4.578
97	9409	912673	9.8488578	4.594
98	9604	941192	9.8994949	4.610
99	9801	970299	9.9498744	4.626
100			0	4.64
101			6	4.65
102			9	4.67
103			6	4.68
104			0	4.70
105			8	4.71
106			1	4.73
107			4	4.74
108			8	4.76
[illegible]			6	4.77
[illegible]			15	4.79
[illegible]			38	4.80
[illegible]			32	4.82
[illegible]			58	4.83
[illegible]			83	4.84
[illegible]			53	4.86
[illegible]			96	4.87
[illegible]			38	4.88
[illegible]			05	4.90
[illegible]			21	4.9
[illegible]			10.9544512	4.9
[illegible]			11.0000000	4.9
[illegible]	14400	1728000	11.0453610	4.9
[illegible]	14641	1771561	11.0905365	4.9
[illegible]	14884	1815848	11.1355287	4.9
[illegible]	15129	1860867		
[illegible]	15376	1906624		

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
125	15625	1953125	11.1803399	5.0000000	.008000000
126	15876	2000376	11.2249722	5.0132979	.007936508
127	16129	2048383	11.2694277	5.0265257	.007874016
128	16384	2097152	11.3137085	5.0396842	.007812500
129	16641	2146689	11.3578167	5.0527743	.007751938
130	16900	2197000	11.4017543	5.0657970	.007692308
131	17161	2248091	11.4455231	5.0787531	.007633588
132	17424	2299968	11.4891253	5.0916434	.007575758
133	17689	2352637	11.5325626	5.1044687	.007518797
134	17956	2406104	11.5758369	5.1172299	.007462687
135	18225	2460375	11.6189500	5.1299278	.007407407
136	18496	2515456	11.6619038	5.1425632	.007352941
137	18769	2571353	11.7046999	5.1551367	.007299270
138	19044	2628072	11.7473401	5.1670493	.007246377
139	19321	2685619	11.7898261	5.1801015	.007194245
140	19600	2744000	11.8321596	5.1924941	.007142857
141	19381	2803221	11.8743421	5.2048279	.007092199
142	20164	2863288	11.9163753	5.2171034	.007042254
143	20449	2924207	11.9582607	5.2293215	.006993007
144	20736	2985984	12.0000000	5.2414828	.006944444
145	21025	3048625	12.0415946	5.2535879	.006896552
146	21316	3112136	12.0830460	5.2656374	.006849315
147	21609	3176523	12.1243557	5.2776321	.006802721
148	21904	3241792	12.1655251	5.2895725	.006756757
149	22201	3307949	12.2065556	5.3014592	.006711409
150	22500	3375000	12.2474487	5.3132928	.006666667
151	22801	3442951	12.2882057	5.3250740	.006622517
152	23104	3511808	12.3288280	5.3368033	.006578947
153	23409	3581577	12.3693169	5.3484812	.006535948
154	23716	3652264	12.4096736	5.3601084	.006493506
155	24025	3723875	12.4498996	5.3716854	.006451613
156	24336	3796416	12.4899960	5.3832126	.006410256
157	24649	3869893	12.5299641	5.3946907	.006369427
158	24964	3944312	12.5698051	5.4061202	.006329114
159	25281	4019679	12.6095202	5.4175015	.006289308
160	25600	4096000	12.6491106	5.4288352	.006250000
161	25921	4173281	12.6385775	5.4401218	.006211180
162	26244	4251528	12.7279221	5.4513618	.006172840
163	26569	4330747	12.7671453	5.4625556	.006134969
164	26896	4410944	12.8062485	5.4737037	.006097561
165	27225	4492125	12.8452326	5.4848066	.006060606
166	27556	4574296	12.8840987	5.4958647	.006024096
167	27889	4657463	12.9228480	5.5068784	.005988024
168	28224	4741632	12.9614814	5.5178484	.005952381
169	28561	4826809	13.0000000	5.5287748	.005917160
170	28900	4913000	13.0384048	5.5396583	.005882353
171	29241	5000211	13.0766968	5.5504991	.005847953
172	29584	5088448	13.1148770	5.5612978	.005813953
173	29929	5177717	13.1529464	5.5720546	.005780347
174	30276	5268024	13.1909060	5.5827702	.005747126
175	30625	5359375	13.2287566	5.5934447	.005714286
176	30976	5451776	13.2664992	5.6040787	.005681818
177	31329	5545233	13.3041347	5.6146724	.005649718
178	31684	5639752	13.3416641	5.6252263	.005617978
179	32041	5735339	13.3790882	5.6357408	.005586592
180	32400	5832000	13.4164079	5.6462162	.005555556
181	32761	5929741	13.4536240	5.6566528	.005524862
182	33124	6028568	13.4907376	5.6670511	.005494505
183	33489	6128487	13.5277493	5.6774114	.005464481
184	33856	6229504	13.5646600	5.6877340	.005434783
185	34225	6331625	13.6014705	5.6980192	.005405405
186	34596	6434856	13.6381817	5.7082675	.005376344

Numbers	Squares	Cubes	Square roots
187	34969	6539203	13.6747943
188	35344	6644672	13.7113092
189	35721	6751269	13.7477271
190	36100	6859000	13.7840488
191	36481	6967871	13.8202750
192	36864	7077888	13.8564065
193	37249	7189057	13.8924440
194	37636	7301384	13.9283883
195	38025	7414875	13.9642400
196	38416	7529536	14.0000000
197	38809	7645373	14.0356688
198	39204	7762392	14.0712473
199	39601	7880599	14.1067360
200	40000	8000000	14.1421356
201	40401	8120601	14.1774469
202	40804	8242408	14.2126704
203	41209	8365427	14.2478068
204	41616	8489664	14.2828569
205	42025	8615125	14.3178211
206	42436	8741816	14.3527001
207	42849	8869743	14.3874946
208	43264	8998912	14.4222051
209	43681	9129329	14.4568323
210	44100	9261000	14.4913767
211	44521	9393931	14.5258390
212	44944	9528128	14.5602198
213	45369	9663597	14.5945195
214	45796	9800344	14.6287388
215	46225	9938375	14.6628783
216	46656	10077696	14.6969385
217	47089	10218313	14.7309199
218	47524	10360232	14.7648231
219	47961	10503459	14.7986486
220	48400	10648000	14.8323970
221	48841	10793861	14.8660687
222	49284	10941048	14.8996644
223	49729	11089567	14.9331845
224	50176	11239424	14.9666295
225	50625	11390625	15.0000000
226	51076	11543176	15.0332964
227	51529	11697083	15.0665192
228	51984	11852352	15.0996689
229	52441	12008989	15.1327460
230	52900	[illegible]	15.1657509
231	53361	[illegible]	15.1986842
232	53824	[illegible]	15.2315462
233	54289	[illegible]	15.2643375
234	54756	[illegible]	15.2970585
235	55225		15.3297097
236	55696		15.3622915
237	56169		15.3948043
238	56644		15.4272486
239	57121		15.4596248
240	57600		15.4919334
241	58081		15.5241747
242	58564		15.5563492
243	59049		15.5884573
244	59536		15.6204994
245	60025		15.6524758
246	60516		15.6843871
247	61009		15.7162336
248	61504		15.7480157

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
249	62001	15438249	15.7797338	6.2911946	.004016064
250	62500	15625000	15.8113883	6.2996053	.004000000
251	63001	15813251	15.8429795	6.3079935	.003984064
252	63504	16003008	15.8745079	6.3163596	.003968254
253	64009	16194277	15.9059737	6.3247035	.003952569
254	64516	16387064	15.9373775	6.3330256	.003937008
255	65025	16581375	15.9687194	6.3413257	.003921569
256	65536	16777216	16.0000000	6.3496042	.003906250
257	66049	16974593	16.0312195	6.3578611	.003891051
258	66564	17173512	16.0623784	6.3660968	.003875969
259	67081	17373979	16.0934769	6.3743111	.003861004
260	67600	17576000	16.1245155	6.3825043	.003846154
261	68121	17779581	16.1554944	6.3906765	.003831418
262	68644	17984728	16.1864141	6.3988279	.003816794
263	69169	18191447	16.2172747	6.4069585	.003802281
264	69696	18399744	16.2480768	6.4150687	.003787879
265	70225	18609625	16.2788206	6.4231583	.003773585
266	70756	18821096	16.3095064	6.4312276	.003759398
267	71289	19034163	16.3401346	6.4392767	.003745318
268	71824	19248832	16.3707055	6.4473057	.003731343
269	72361	19465109	16.4012195	6.4553148	.003717472
270	72900	19683000	16.4316767	6.4633041	.003703704
271	73441	19902511	16.4620776	6.4712736	.003690037
272	73984	20123648	16.4924225	6.4792236	.003676471
273	74529	20346417	16.5227116	6.4871541	.003663004
274	75076	20570824	16.5529454	6.4950653	.003649635
275	75625	20796875	16.5831240	6.5029572	.003636364
276	76176	21024576	16.6132477	6.5108300	.003623188
277	76729	21253933	16.6433170	6.5186839	.003610108
278	77284	21484952	16.6733320	6.5265189	.003597122
279	77841	21717639	16.7032931	6.5343351	.003584229
280	78400	21952000	16.7332005	6.5421326	.003571429
281	78961	22188041	16.7630546	6.5499116	.003558719
282	79524	22425768	16.7928553	6.5576722	.003546099
283	80089	22665187	16.8226038	6.5654144	.003533569
284	80656	22906304	16.8522995	6.5731385	.003521127
285	81225	23149125	16.8819430	6.5808443	.003508772
286	81796	23393656	16.9115345	6.5885323	.003496503
287	82369	23639903	16.9410743	6.5962023	.003484321
288	82944	23887872	16.9705627	6.6038545	.003472222
289	83521	24137569	17.0000000	6.6114890	.003460208
290	84100	24389000	17.0293864	6.6191060	.003448276
291	84681	24642171	17.0587221	6.6267054	.003436426
292	85264	24897088	17.0880075	6.6342874	.003424658
293	85849	25153757	17.1172428	6.6418522	.003412969
294	86436	25412184	17.1464282	6.6493998	.003401361
295	87025	25672375	17.1755640	6.6569302	.003389831
296	87616	25934336	17.2046505	6.6644437	.003378378
297	88209	26198073	17.2336879	6.6719403	.003367003
298	88804	26463592	17.2626765	6.6794200	.003355705
299	89401	26730899	17.2916165	6.6868831	.003344482
300	90000	27000000	17.3205081	6.6943295	.003333333
301	90601	27270901	17.3493516	6.7017593	.003322259
302	91204	27543608	17.3781472	6.7091729	.003311258
303	91809	27818127	17.4068952	6.7165700	.003300330
304	92416	28094464	17.4355958	6.7239508	.003289474
305	93025	28372625	17.4642492	6.7313155	.003278689
306	93636	28652616	17.4928557	6.7386641	.003267974
307	94249	28934443	17.5214155	6.7459967	.003257329
308	94864	29218112	17.5499288	6.7533134	.003246753
309	95481	29503629	17.5783958	6.7606143	.003236246
310	96100	29791000	17.6068169	6.7678995	.003225806

			Square roots	Cube roots
311	96721	30080231	17.6351921	6.77516
312	97344	30371328	17.6635217	6.78342
313	97969	30664297	17.6918060	6.78966
314	98596	30959144	17.7200451	6.79688
315	99225	31255875	17.7482393	6.80409
316	99856	31554496	17.7763888	6.81128
317	100489	31855013	17.8044938	6.81846
318	101124	32157432	17.8325545	6.82562
319	101761	32461759	17.8605711	6.83277
320	102400	32768000	17.8885438	6.83990
321	103041	33076161	17.9164729	6.847021
322	103684	33386248	17.9443584	6.854124
323	104329	33698267	17.9722008	6.861212
324	104976	34012224	18.0000000	6.868285
325	105625	34328125	18.0277564	6.875344
326	106276	34645976	18.0554701	6.882388
327	106929	34965783	18.0831413	6.889419
328	107584	35287552	18.1107703	6.896434
329	108241	35611289	18.1383571	6.903436
330	108900	35937000	18.1659021	6.910423
331	109561	36264691	18.1934054	6.917396
332	110224	36594368	18.2208672	6.924355
333	110889	36926037	18.2482876	6.931300
334	111556	37259704	18.2756669	6.938232
335	112225	37595375	18.3030052	6.945149
336	112896	37933056	18.3303028	6.952053
337	113569	38272753	18.3575598	6.958943
338	114244	38614472	18.3847763	6.965819
339	114921	38958219	18.4119526	6.972682
340	115600	39304000	18.4390889	6.979532
341	116281	39651821	18.4661853	6.986368
342	116964	40001688	18.4932420	6.993190
343	117649	40353607	18.5202592	7.000000
344	118336	40707584	18.5472370	7.006796
345	119025	41063625	18.5741756	7.013579
346	119716	41421736	18.6010752	7.020349
347	120409	41781923	18.6279360	7.027105
348	121104	42144192	18.6547581	7.033849
349	121801	42508549	18.6815417	7.040580
350	122500	42875000	18.7082869	7.047298
351	123201	43243551	18.7349940	7.054004
352	123904	43614208	18.7616630	7.060696
353	124609	43986977	18.7882942	7.067376
354	125316	44361864	18.8148877	7.074044
355	126025	44738875	18.8414437	7.080699
356	126736	45118016	18.8679623	7.087341
357	127449	45499293	18.8944436	7.093970
358	128164	45882712	18.9208879	7.100588
359	128881	46268279	18.9472953	7.107193
360	129600	46656000	18.9736660	7.113786
361	130321	47045881	19.0000000	7.120367
362	131044	47437928	19.0262976	7.126936
363	131769	47832147	19.0525589	7.133492
364	132496	48228544	19.0787840	7.140037
365	133225	48627125	19.1049732	7.146569
366	133956	49027896	19.1311265	7.153090
367	134689	49430863	19.1572441	7.159599
368	135424	49836032	19.1833261	7.166095
369	136161	50243409	19.2093727	7.172580
370	136900	50653000	19.2353841	7.179054
371	137641	51064811	19.2613603	7.185516
372	[illegible]	51478848	19.2873015	7.191966

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
373	139129	51895117	19.3132079	7.1984050	.002680965
374	139876	52313624	19.3390796	7.2048322	.002673797
375	140625	52734375	19.3649167	7.2112479	.002666667
376	141376	53157376	19.3907194	7.2176522	.002659574
377	142129	53582633	19.4164878	7.2240450	.002652520
378	142884	54010152	19.4422221	7.2304268	.002645503
379	143641	54439939	19.4679223	7.2367972	.002638522
380	144400	54872000	19.4935887	7.2431565	.002631579
381	145161	55306341	19.5192213	7.2495045	.002624672
382	145924	55742968	19.5448203	7.2558415	.002617801
383	146689	56181887	19.5703858	7.2621675	.002610966
384	147456	56623104	19.5959179	7.2684824	.002604167
385	148225	57066625	19.6214169	7.2747864	.002597403
386	148996	57512456	19.6468827	7.2810794	.002590674
387	149769	57960603	19.6723156	7.2873617	.002583979
388	150544	58411072	19.6977156	7.2936330	.002577320
389	151321	58863869	19.7230829	7.2998936	.002570694
390	152100	59319000	19.7484177	7.3061436	.002564103
391	152881	59776471	19.7737199	7.3123828	.002557545
392	153664	60236288	19.7989899	7.3186114	.002551020
393	154449	60698457	19.8242276	7.3248295	.002544529
394	155236	61162984	19.8494332	7.3310369	.002538071
395	156025	61629875	19.8746069	7.3372339	.002531646
396	156816	62099136	19.8997487	7.3434205	.002525253
397	157609	62570773	19.9248588	7.3495966	.002518892
398	158404	63044792	19.9499373	7.3557624	.002512563
399	159201	63521199	19.9749844	7.3619178	.002506266
400	160000	64000000	20.0000000	7.3680630	.002500000
401	160801	64481201	20.0249844	7.3741979	.002493766
402	161604	64964808	20.0499377	7.3803227	.002487562
403	162409	65450827	20.0748599	7.3864373	.002481390
404	163216	65939264	20.0997512	7.3925418	.002475248
405	164025	66430125	20.1246118	7.3986363	.002469136
406	164836	66923416	20.1494417	7.4047206	.002463054
407	165649	67419143	20.1742410	7.4107950	.002457002
408	166464	67917312	20.1990099	7.4168595	.002450980
409	167281	68417929	20.2237484	7.4229142	.002444988
410	168100	68921000	20.2484567	7.4289589	.002439024
411	168921	69426531	20.2731349	7.4349938	.002433090
412	169744	69934528	20.2977831	7.4410189	.002427184
413	170569	70444997	20.3224014	7.4470342	.002421308
414	171396	70957944	20.3469899	7.4530399	.002415459
415	172225	71473375	20.3715488	7.4590359	.002409639
416	173056	71991296	20.3960781	7.4650223	.002403846
417	173889	72511713	20.4205779	7.4709991	.002398082
418	174724	73034632	20.4450483	7.4769664	.002392344
419	175561	73560059	20.4694895	7.4829242	.002386635
420	176400	74088000	20.4939015	7.4888724	.002380952
421	177241	74618461	20.5182845	7.4948113	.002375297
422	178084	75151448	20.5426386	7.5007406	.002369668
423	178929	75686967	20.5669638	7.5066607	.002364066
424	179776	76225024	20.5912603	7.5125715	.002358491
425	180625	76765625	20.6155281	7.5184730	.002352941
426	181476	77308776	20.6397674	7.5243652	.002347418
427	182329	77854483	20.6639783	7.5302482	.002341920
428	183184	78402752	20.6881609	7.5361221	.002336449
429	184041	78953589	20.7123152	7.5419867	.002331002
430	184900	79507000	20.7364414	7.5478423	.002325581
431	185761	80062991	20.7605395	7.5536888	.002320186
432	186624	80621568	20.7846097	7.5595263	.002314815
433	187489	81182737	20.8086520	7.5653548	.002309469
434	188356	81746504	20.8326667	7.5711743	.002304147

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
435	189225	82312875	20.8566536	7.5769849	.00229885
436	190096	82881856	20.8806130	7.5827865	.00229357
437	190969	83453453	20.9045450	7.5885793	.00228833
438	191844	84027672	20.9284495	7.5943633	.00228310
439	192721	84604519	20.9523268	7.6001385	.00227790
440	193600	85184000	20.9761770	7.6059049	.00227272
441	194481	85766121	21.0000000	7.6116626	.00226757
442	195364	86350888	21.0237960	7.6174116	.00226244
443	196249	86938307	21.0475652	7.6231519	.00225733
444	197136	87528384	21.0713075	7.6288837	.00225225
445	198025	88121125	21.0950231	7.6346067	.00224719
446	198916	88716536	21.1187121	7.6403213	.00224215
447	199809	89314623	21.1423745	7.6460272	.00223713
448	200704	89915392	21.1660105	7.6517247	.00223214
449	201601	90518849	21.1896201	7.6574138	.00222717
450	202500	91125000	21.2132034	7.6630943	.00222222
451	203401	91733851	21.2367606	7.6687665	.00221729
452	204304	92345408	21.2602916	7.6744303	.00221238
453	205209	92959677	21.2837967	7.6800857	.00220750
454	206116	93576664	21.3072758	7.6857328	.00220264
455	207025	94196375	21.3307290	7.6913717	.00219780
456	207936	94818816	21.3541565	7.6970023	.00219298
457	208849	95443993	21.3775583	7.7026246	.00218818
458	209764	96071912	21.4009346	7.7082388	.00218340
459	210681	96702579	21.4242853	7.7138448	.00217864
460	211600	97336000	21.4476106	7.7194426	.00217391
461	212521	97972181	21.4709106	7.7250325	.00216919
462	213444	98611128	21.4941853	7.7306141	.00216450
463	214369	99252847	21.5174348	7.7361877	.00215982
464	215296	99897344	21.5406592	7.7417532	.00215517
465	216225	100544625	21.5638587	7.7473109	.00215053
466	217156	101194696	21.5870331	7.7528606	.00214592
467	218089	101847563	21.6101828	7.7584023	.00214132
468	219024	102503232	21.6333077	7.7639361	.00213675
469	219961	103161709	21.6564078	7.7694620	.00213219
470	220900	103823000	21.6794834	7.7749801	.00212766
471	221841	104487111	21.7025344	7.7804904	.00212314
472	222784	105154048	21.7255610	7.7859928	.00211864
473	223729	105823817	21.7485632	7.7914875	.00211416
474	224676	106496424	21.7715411	7.7969745	.00210970
475	225625	107171875	21.7944947	7.8024538	.00210526
476	226576	107850176	21.8174242	7.8079254	.00210084
477	227529	108531333	21.8403297	7.8133892	.00209643
478	228484	109215352	21.8632111	7.8188456	.00209205
479	229441	109902239	21.8860686	7.8242942	.00208768
480	230400	110592000	21.9089023	7.8297353	.00208333
481	231361	111284641	21.9317122	7.8351688	.00207900
482	232324	111980168	21.9544984	7.8405949	.00207468
483	233289	112678587	21.9772610	7.8460134	.00207039
484	234256	113379904	22.0000000	5.8514244	.00206611
485	235225	114084125	22.0227155	7.8568281	.00206185
486	236196	114791256	22.0454077	7.8622242	.00205761
487	237169	115501303	22.0680765	7.8676130	.00205338
488	238144	116214272	22.0907220	7.8729944	.00204918
489	239121	116930169	22.1133444	7.8783684	.00204499
490	240100	117649000	22.1359436	7.8837352	.00204081
491	241081	118370771	22.1585198	7.8890946	.00203666
492	242064	119095488	22.1810730	7.8944468	.00203252
493	243049	119823157	22.2036033	7.8997917	.00202839
494	244036	120553784	22.2261108	7.9051294	.00202429
495	245025	121287375	22.2485955	7.9104599	.00202020
496	246016	122023936	22.2710575	7.9157832	.00201612

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
497	247009	122763473	22.2934968	7.9210994	.002012072
498	248004	123505992	22.3159136	7.9264085	.002008032
499	249001	124251499	22.3383079	7.9317104	.002004008
500	250000	125000000	22.3606798	7.9370053	.002000000
501	251001	125751501	22.3830293	7.9422931	.001996008
502	252004	126506008	22.4053565	7.9475739	.001992032
503	253009	127263527	22.4276615	7.9528477	.001988072
504	254016	128024064	22.4499443	7.9581144	.001984127
505	255025	128787625	22.4722051	7.9633743	.001980198
506	256036	129554216	22.4944438	7.9686271	.001976285
507	257049	130323843	22.5166605	7.9738731	.001972387
508	258064	131096512	22.5388553	7.9791122	.001968504
509	259081	131872229	22.5610283	7.9843444	.001964637
510	260100	132651000	22.5831796	7.9895697	.001960784
511	261121	133432831	22.6053091	7.9947883	.001956947
512	262144	134217728	22.6274170	8.0000000	.001953125
513	263169	135005697	22.6495033	8.0052049	.001949318
514	264196	135796744	22.6715681	8.0104032	.001945525
515	265225	136590875	22.6936114	8.0155946	.001941748
516	266256	137388096	22.7156334	8.0207794	.001937984
517	267289	138188413	22.7376340	8.0259574	.001934236
518	268324	138991832	22.7596134	8.0311287	.001930502
519	269361	139798359	22.7815715	8.0362935	.001926782
520	270400	140608000	22.8035085	8.0414515	.001923077
521	271441	141420761	22.8254244	8.0466030	.001919386
522	272484	142236648	22.8473193	8.0517479	.001915709
523	273529	143055667	22.8691933	8.0568862	.001912046
524	274576	143877824	22.8910463	8.0620180	.001908397
525	275625	144703125	22.9128785	8.0671432	.001904762
526	276676	145531576	22.9346899	8.0722620	.001901141
527	277729	146363183	22.9564806	8.0773743	.001897533
528	278784	147197952	22.9782506	8.0824800	.001893939
529	279841	148035889	23.0000000	8.0875794	.001890359
530	280900	148877000	23.0217289	8.0926723	.001886792
531	281961	149721291	23.0434372	8.0977589	.001883239
532	283024	150568768	23.0651252	8.1028390	.001879699
533	284089	151419437	23.0867928	8.1079128	.001876173
534	285156	152273304	23.1084400	8.1129803	.001872659
535	286225	153130375	23.1300670	8.1180414	.001869159
536	287296	153990656	23.1516738	8.1230962	.001865672
537	288369	154854153	23.1732605	8.1281447	.001862197
538	289444	155720872	23.1948270	8.1331870	.001858736
539	290521	156590819	23.2163735	8.1382230	.001855288
540	291600	157464000	23.2379001	8.1432529	.001851852
541	292681	158340421	23.2594067	8.1482765	.001848429
542	293764	159220088	23.2808935	8.1532939	.001845018
543	294849	160103007	23.3023604	8.1583051	.001841621
544	295936	160989184	23.3238076	8.1633102	.001838235
545	297025	161878625	23.3452351	8.1683092	.001834862
546	298116	162771336	23.3666429	8.1733020	.001831502
547	299209	163667323	23.3880311	8.1782888	.001828154
548	300304	164566592	23.4093998	8.1832695	.001824818
549	301401	165469149	23.4307490	8.1882441	.001821494
550	302500	166375000	23.4520788	8.1932127	.001818182
551	303601	167284151	23.4733892	8.1981753	.001814882
552	304704	168196608	23.4946802	8.2031319	.001811594
553	305809	169112377	23.5159520	8.2080825	.001808318
554	306916	170031464	23.5372046	8.2130271	.001805054
555	308025	170953875	23.5584380	8.2179657	.001801802
556	309136	171879616	23.5796522	8.2228985	.001798561
557	310249	172808693	23.6008474	8.2278254	.001795332
558	311364	173741112	23.6220236	8.2327463	.001792115

559	312481	174676879	23.6431808	8.2376614
560	313600	175616000	23.6643191	8.2425706
561	314721	176558481	23.6854386	8.2474740
562	315844	177504328	23.7065392	8.2523715
563	316969	178453547	23.7276210	8.2572633
564	318096	179406144	23.7486842	8.2621492
565	319225	180362125	23.7697286	8.2670294
566	320356	181321496	23.7907545	8.2719038
567	321489	182284263	23.8117618	8.2767725
568	322624	183250432	23.8327506	8.2816355
569	323761	184220009	23.8537209	8.2864928
570	324900	185193000	23.8746728	8.2913443
571	326041	186169411	23.8956063	8.2961902
572	327184	187149248	23.9165215	8.3010305
573	328329	188132517	23.9374184	8.3058651
574	329476	189119224	23.9582971	8.3106941
575	330625	190109375	23.9791576	8.3155175
576	331776	191102976	24.0000000	8.3203353
577	332929	192100033	24.0208243	8.3251475
578	334084	193100552	24.0416306	8.3299542
579	335241	194104539	24.0624188	8.3347553
580	336400	195112000	24.0831891	8.3395509
581	337561	196122941	24.1039416	8.3443410
582	338724	197137368	24.1246762	8.3491256
583	339889	198155287	24.1453929	8.3539047
584	341056	199176704	24.1660919	8.3586784
585	342225	200201625	24.1867732	8.3634466
586	343396	201230056	24.2074369	8.3682094
587	344569	202262003	24.2280829	8.3729668
588	345744	203297472	24.2487113	8.3777187
589	346921	204336469	24.2693222	8.3824653
590	348100	205379000	24.2899156	8.3872063
591	349281	206425071	24.3104916	8.3919424
592	350464	207474688	24.3310501	8.3966729
593	351649	208527857	24.3515913	8.4013981
594	352836	209584584	24.3721152	8.4061180
595	354025	210644875	24.3926218	8.4108326
596	355216	211708736	24.4131112	8.4155419
597	356409	212776173	24.4335834	8.4202459
598	357604	213847192	24.4540385	8.4249447
599	358801	214921799	24.4744765	8.4296383
600	360000	216000000	24.4948974	8.4343267
601	361201	217081801	24.5153013	8.4390098
602	362404	218167208	24.5356883	8.4436877
603	363609	219256227	24.5560583	8.4483604
604	364816	220348864	24.5764115	8.4530281
605	366025	221445125	24.5967478	8.4576906
606	367236	222545016	24.6170673	8.4623479
607	368449	223648543	24.6373700	8.4670001
608	369664	224755712	24.6576560	8.4716472
609	370881	225866529	24.6779254	8.4762892
610	372100	226981000	24.6981781	8.4809261
611	373321	228099131	24.7184142	8.4855579
612	374544	229220928	24.7386338	8.4901847
613	375769	230346397	24.7588368	8.4948065
614	376996	231475544	24.7790234	8.4994233
615	378225	232608375	24.7991935	8.5040350
616	379456	233744896	24.8193473	8.5086417
617	380689	234885113	24.8394847	8.5132435
618	381924	236029032	24.8596058	8.5178403
619	383161	237176659	24.8797106	8.5224321
620	384400	238328000	24.8997992	8.5270190

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
621	385641	239483061	24.9198716	8.5316009	.001610306
622	386884	240641848	24.9399278	8.5361780	.001607717
623	388129	241804367	24.9599679	8.5407501	.001605136
624	389376	242970624	24.9799920	8.5453173	.001602564
625	390625	244140625	25.0000000	8.5498797	.001600000
626	391876	245314376	25.0199920	8.5544372	.001597444
627	393129	246491883	25.0399681	8.5589899	.001594896
628	394384	247673152	25.0599282	8.5635377	.001592357
629	395641	248858189	25.0798724	8.5680807	.001589825
630	396900	250047000	25.0998008	8.5726189	.001587302
631	398161	251239591	25.1197134	8.5771523	.001584786
632	399424	252435968	25.1396102	8.5816809	.001582278
633	400689	253636137	25.1594913	8.5862047	.001579779
634	401956	254840104	25.1793566	8.5907238	.001577287
635	403225	256047875	25.1992063	8.5952380	.001574803
636	404496	257259456	25.2190404	8.5997476	.001572327
637	405769	258474853	25.2388589	8.6042525	.001569859
638	407044	259694072	25.2586619	8.6087526	.001567398
639	408321	260917119	25.2784493	8.6132480	.001564945
640	409600	262144000	25.2982213	8.6177388	.001562500
641	410881	263374721	25.3179778	8.6222248	.001560062
642	412164	264609288	25.3377189	8.6267063	.001557632
643	413449	265847707	25.3574447	8.6311830	.001555210
644	414736	267089984	25.3771551	8.6356551	.001552795
645	416025	268336125	25.3968502	8.6401226	.001550388
646	417316	269586136	25.4165301	8.6445855	.001547988
647	418609	270840023	25.4361947	8.6490437	.001545595
648	419904	272097792	25.4558441	8.6534974	.001543210
649	421201	273359449	25.4754784	8.6579465	.001540832
650	422500	274625000	25.4950976	8.6623911	.001538462
651	423801	275894451	25.5147016	8.6668310	.001536098
652	425104	277167808	25.5342907	8.6712665	.001533742
653	426409	278445077	25.5538647	8.6756974	.001531394
654	427716	279726264	25.5734237	8.6801237	.001529052
655	429025	281011375	25.5929678	8.6845456	.001526718
656	430336	282300416	25.6124969	8.6889630	.001524390
657	431649	283593393	25.6320112	8.6933759	.001522070
658	432964	284890312	25.6515107	8.6977843	.001519757
659	434281	286191179	25.6709953	8.7021882	.001517451
660	435600	287496000	25.6904652	8.7065877	.001515152
661	436921	288804781	25.7099203	8.7109827	.001512859
662	438244	290117528	25.7293607	8.7153734	.001510574
663	439569	291434247	25.7487864	8.7197596	.001508296
664	440896	292754944	25.7681975	8.7241414	.001506024
665	442225	294079625	25.7875939	8.7285187	.001503759
666	443556	295408296	25.8069758	8.7328918	.001501502
667	444889	296740963	25.8263431	8.7372604	.001499250
668	446224	298077632	25.8456960	8.7416246	.001497006
669	447561	299418309	25.8650343	8.7459846	.001494768
670	448900	300763000	25.8843582	8.7503401	.001492537
671	450241	302111711	25.9036677	8.7546913	.001490313
672	451584	303464448	25.9229628	8.7590383	.001488095
673	452929	304821217	25.9422435	8.7633809	.001485884
674	454276	306182024	25.9615100	8.7677192	.001483680
675	455625	307546875	25.9807621	8.7720532	.001481481
676	456976	308915776	26.0000000	8.7763830	.001479290
677	458329	310288733	26.0192237	8.7807084	.001477105
678	459684	311665752	26.0384331	8.7850296	.001474926
679	461041	313046839	26.0576284	8.7893466	.004472754
680	462400	314432000	26.0768096	8.7936593	.001470588
681	463761	315821241	26.0959767	8.7979679	.001468429
682	465124	317214568	26.1151297	8.8022721	.001466276

No.	Squares	Cubes	Square roots	Cube roots
683	466489	318611987	26.1342687	8.8065722
684	467856	320013504	26.1533937	8.8108681
685	469225	321419125	26.1725047	8.8151598
686	470596	322828856	26.1916017	8.8194474
687	471969	324242703	26.2106848	8.8237307
688	473344	325660672	26.2297541	8.8280099
689	474721	327082769	26.2488095	8.8322850
690	476100	328509000	26.2678511	8.8365559
691	477481	329939371	26.2868789	8.8408227
692	478864	331373888	26.3058929	8.8450854
693	480249	332812557	26.3248932	8.8493440
694	481636	334255384	26.3438797	8.8535985
695	483025	335702375	26.3628527	8.8578489
696	484416	337153536	26.3818119	8.8620952
697	485809	338608873	26.4007576	8.8663375
698	487204	340068392	26.4196896	8.8705757
699	488601	341532099	26.4386081	8.8748099
700	490000	343000000	26.4575131	8.8790400
701	491401	344472101	26.4764046	8.8832661
702	492804	345948408	26.4952826	8.8874882
703	494209	347428927	26.5141472	8.8917063
704	495616	348913664	26.5329983	8.8959204
705	497025	350402625	26.5518361	8.9001304
706	498436	351895816	26.5706605	8.9043366
707	499849	353393243	26.5894716	8.9085387
708	501264	354894912	26.6082694	8.9127369
709	502681	356400829	26.6270539	8.9169311
710	504100	357911000	26.6458252	8.9211214
711	505521	359425431	26.6645833	8.9253078
712	506944	360944128	26.6833281	8.9294902
713	508369	362467097	26.7020598	8.9336687
714	509796	363994344	26.7207784	8.9378433
715	511225	365525875	26.7394839	8.9420140
716	512656	367061696	26.7581763	8.9461809
717	514089	368601813	26.7768557	8.9503438
718	515524	370146232	26.7955220	8.9545029
719	516961	371694959	26.8141754	8.9586581
720	518400	373248000	26.8328157	8.9628095
721	519841	374805361	26.8514432	8.9669570
722	521284	376367048	26.8700577	8.9711007
723	522729	377933067	26.8886593	8.9752406
724	524176	379503424	26.9072481	8.9793766
725	525625	381078125	26.9258240	8.9835089
726	527076	382657176	26.9443872	8.9876373
727	528529	384240583	26.9629375	8.9917620
728	529984	385828352	26.9814751	8.9958829
729	531441	387420489	27.0000000	9.0000000
730	532900	389017000	27.0185122	9.0041134
731	534361	390617891	27.0370117	9.0082229
732	535824	392223168	27.0554985	9.0123288
733	537289	393832837	27.0739727	9.0164309
734	538756	395446904	27.0924344	9.0205293
735	540225	397065375	27.1108834	9.0246239
736	541696	398688256	27.1293199	9.0287149
737	543169	400315553	27.1477439	9.0328021
738	544644	401947272	27.1661554	9.0368857
739	546121	403583419	27.1845544	9.0409655
740	547600	405224000	27.2029410	9.0450419
741	549081	406869021	27.2213152	9.0491142
742	550564	408518488	27.2396769	9.0531831
743	552049	410172407	27.2580263	9.0572482

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
745	[illegible]	[illegible]	27.2946881	[illegible]	[illegible]
746	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
747	[illegible]	[illegible]	[illegible]	9.0734726	[illegible]
748	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
749	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
750	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
751	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
752	[illegible]	[illegible]	27.4226184	[illegible]	[illegible]
753	[illegible]	[illegible]	27.4408454	[illegible]	[illegible]
754	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
755	[illegible]	[illegible]	27.4772633	[illegible]	[illegible]
756	571536	[illegible]	[illegible]	[illegible]	[illegible]
757	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
758	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
759	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
760	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
761	579121	440711081	[illegible]	[illegible]	[illegible]
762	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
763	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
764	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
765	[illegible]	447697125	27.6586334	[illegible]	[illegible]
766	[illegible]	[illegible]	27.6767050	[illegible]	[illegible]
767	[illegible]	451217663	27.6947648	[illegible]	[illegible]
768	[illegible]	[illegible]	27.7128129	[illegible]	[illegible]
769	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
770	[illegible]	[illegible]	27.7488739	[illegible]	[illegible]
771	[illegible]	[illegible]	27.7668868	[illegible]	.00129702
772	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
773	[illegible]	461889917	[illegible]	[illegible]	[illegible]
774	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
775	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
776	[illegible]	[illegible]	[illegible]	9.1864018	[illegible]
777	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
778	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
779	[illegible]	[illegible]	27.9106715	[illegible]	[illegible]
780	[illegible]	474552000	[illegible]	[illegible]	[illegible]
781	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
782	[illegible]	478211768	[illegible]	[illegible]	[illegible]
783	[illegible]	[illegible]	27.9821372	[illegible]	[illegible]
784	[illegible]	481890304	[illegible]	[illegible]	[illegible]
785	[illegible]	[illegible]	28.0178515	[illegible]	[illegible]
786	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
787	[illegible]	[illegible]	28.0535203	[illegible]	[illegible]
788	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
789	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
790	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
791	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
792	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
793	[illegible]	[illegible]	28.1602557	[illegible]	[illegible]
794	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
795	[illegible]	[illegible]	28.1957444	[illegible]	[illegible]
796	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
797	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
798	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
799	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
800	[illegible]	512000000	[illegible]	[illegible]	[illegible]
801	[illegible]	513922401	[illegible]	[illegible]	[illegible]
802	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
803	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
804	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
805	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
806	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
807	651249	525557943	28.4077454	9.3101750	.001239157
808	652864	527514112	28.4253408	9.3140190	.001237624
809	654481	529475129	28.4429253	9.3178599	.001236094
810	656100	531441000	28.4604989	9.3216975	.001234568
811	657721	533411731	28.4780617	9.3255320	.001233046
812	659344	535387328	28.4956137	9.3293634	.001231527
813	660969	537367797	28.5131549	9.3331916	.001230012
814	662596	539353144	28.5306852	9.3370167	.001228501
815	664225	541343375	28.5482048	9.3408386	.001226994
816	665856	543338496	28.5657137	9.3446575	.001225490
817	667489	545338513	28.5832119	9.3484731	.001223990
818	669124	547343432	28.6006993	9.3522857	.001222494
819	670761	549353259	28.6181760	9.3560952	.001221001
820	672400	551368000	28.6356421	9.3599016	.001219512
821	674041	553387661	28.6530976	9.3637049	.001218027
822	675684	555412248	28.6705424	9.3675051	.001216545
823	677329	557441767	28.6879766	9.3713022	.001215067
824	678976	559476224	28.7054002	9.3750963	.001213592
825	680625	561515625	28.7228132	9.3788873	.001212121
826	682276	563559976	28.7402157	9.3826752	.001210654
827	683929	565609283	28.7576077	9.3864600	.001209190
828	685584	567663552	28.7749891	9.3902419	.001207729
829	687241	569722789	28.7923601	9.3940206	.001206273
830	688900	571787000	28.8097206	9.3977964	.001204819
831	690561	573856191	28.8270706	9.4015691	.001203369
832	692224	575930368	28.8444102	9.4053387	.001201923
833	693889	578009537	28.8617394	9.4091054	.001200480
834	695556	580093704	28.8790582	9.4128690	.001199041
835	697225	582182875	28.8963666	9.4166297	.001197605
836	698896	584277056	28.9136646	9.4203873	.001196172
837	700569	586376263	28.9309523	9.4241420	.001194743
838	702244	588480472	28.9482297	9.4278936	.001193317
839	703921	590589719	28.9654967	9.4316423	.001191895
840	705600	592704000	28.9827535	9.4353880	.001190476
841	707281	594823321	29.0000000	9.4391307	.001189061
842	708964	596947688	29.0172363	9.4428704	.001187648
843	710649	599077107	29.0344623	9.4466072	.001186240
844	712336	601211584	29.0516781	9.4503410	.001184834
845	714025	603351125	29.0688837	9.4540719	.001183432
846	715716	605495736	29.0860791	9.4577999	.001182033
847	717409	607645423	29.1032644	9.4615249	.001180638
848	719104	609800192	29.1204396	9.4652470	.001179245
849	720801	611960049	29.1376046	9.4689661	.001177856
850	722500	614125000	29.1547595	9.4726824	.001176471
851	724201	616295051	29.1719043	9.4763957	.001175088
852	725904	618470208	29.1890390	9.4801061	.001173709
853	727609	620650477	29.2061637	9.4838136	.001172333
854	729316	622835864	29.2232784	9.4875182	.001170960
855	731025	625026375	29.2403830	9.4912200	.001169591
856	732736	627222016	29.2574777	9.4949188	.001168224
857	734449	629422793	29.2745623	9.4986147	.001166861
858	736164	631628712	29.2916370	9.5023078	.001165501
859	737881	633839779	29.3087018	9.5059980	.001164144
860	739600	636056000	29.3257566	9.5096854	.001162791
861	741321	638277381	29.3428015	9.5133699	.001161440
862	743044	640503928	29.3598365	9.5170515	.001160093
863	744769	642735647	29.3768616	9.5207303	.001158749
864	746496	644972544	29.3938769	9.5244063	.001157407
865	748225	647214625	29.4108823	9.5280794	.001156069
866	749956	649461896	29.4278779	9.5317497	.001154734
867	751689	651714363	29.4448637	9.5354172	.001153403

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
249	62001	15438249	15.7797338	6.2911946	.004016064
250	62500	15625000	15.8113883	6.2996053	.004000000
251	63001	15813251	15.8429795	6.3079935	.003984064
252	63504	16003008	15.8745079	6.3163596	.003968254
253	64009	16194277	15.9059737	6.3247035	.003952569
254	64516	16387064	15.9373775	6.3330256	.003937008
255	65025	16581375	15.9687194	6.3413257	.003921569
256	65536	16777216	16.0000000	6.3496042	.003906250
257	66049	16974593	16.0312195	6.3578611	.003891051
258	66564	17173512	16.0623784	6.3660968	.003875969
259	67081	17373979	16.0934769	6.3743111	.003861004
260	67600	17576000	16.1245155	6.3825043	.003846154
261	68121	17779581	16.1554944	6.3906765	.003831418
262	68644	17984728	16.1864141	6.3988279	.003816794
263	69169	18191447	16.2172747	6.4069585	.003802281
264	69696	18399744	16.2480768	6.4150687	.003787879
265	70225	18609625	16.2788206	6.4231583	.003773585
266	70756	18821096	16.3095064	6.4312276	.003759398
267	71289	19034163	16.3401346	6.4392767	.003745318
268	71824	19248832	16.3707055	6.4473057	.003731343
269	72361	19465109	16.4012195	6.4553148	.003717472
270	72900	19683000	16.4316767	6.4633041	.003703704
271	73441	19902511	16.4620776	6.4712736	.003690037
272	73984	20123648	16.4924225	6.4792236	.003676471
273	74529	20346417	16.5227116	6.4871541	.003663004
274	75076	20570824	16.5529454	6.4950653	.003649635
275	75625	20796875	16.5831240	6.5029572	.003636364
276	76176	21024576	16.6132477	6.5108300	.003623188
277	76729	21253933	16.6433170	6.5186839	.003610108
278	77284	21484952	16.6733320	6.5265189	.003597122
279	77841	21717639	16.7032931	6.5343351	.003584229
280	78400	21952000	16.7332005	6.5421326	.003571429
281	78961	22188041	16.7630546	6.5499116	.003558719
282	79524	22425768	16.7928553	6.5576722	.003546099
283	80089	22335187	16.8226038	6.5654144	.003533569
284	80656	22906304	16.8522995	6.5731385	.003521127
285	81225	23149125	16.8819430	6.5808443	.003508772
286	81796	23393655	16.9115345	6.5885323	.003496503
287	82369	23639903	16.9410743	6.5962023	.003484321
288	82944	23887872	16.9705627	6.6038545	.003472222
289	83521	24137569	17.0000000	6.6114890	.003460208
290	84100	24389000	17.0293864	6.6191060	.003448276
291	84681	24642171	17.0587221	6.6267054	.003436426
292	85264	24897088	17.0880075	6.6342874	.003424658
293	85849	25153757	17.1172428	6.6418522	.003412969
294	86436	25412184	17.1464282	6.6493998	.003401361
295	87025	25672375	17.1755640	6.6569302	.003389831
296	87616	25934336	17.2046505	6.6644437	.003378378
297	88209	26198073	17.2336879	6.6719403	.003367003
298	88804	26463592	17.2626765	6.6794200	.003355705
299	89401	26730899	17.2916165	6.6868831	.003344482
300	90000	27000000	17.3205081	6.6943295	.003333333
301	90601	27270901	17.3493516	6.7017593	.003322259
302	91204	27543608	17.3781472	6.7091729	.003311258
303	91809	27818127	17.4068952	6.7165700	.003300330
304	92416	28094464	17.4355958	6.7239508	.003289474
305	93025	28372625	17.4642492	6.7313155	.003278689
306	93636	28652616	17.4928557	6.7386641	.003267974
307	94249	28934443	17.5214155	6.7459967	.003257329
308	94864	29218112	17.5499288	6.7533134	.003246753
309	95481	29503629	17.5783958	6.7606143	.003236246
310	96100	29791000	17.6068169	6.7678995	.003225806

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
373	139129	51895117	19.3132079	7.1984050	.002680965
374	139876	52313624	19.3390796	7.2048322	.002673797
375	140625	52734375	19.3649167	7.2112479	.002666667
376	141376	53157376	19.3907194	7.2176522	.002659574
377	142129	53582633	19.4164878	7.2240450	.002652520
378	142884	54010152	19.4422221	7.2304268	.002645503
379	143641	54439939	19.4679223	7.2367972	.002638522
380	144400	54872000	19.4935887	7.2431565	.002631579
381	145161	55306341	19.5192213	7.2495045	.002624672
382	145924	55742968	19.5448203	7.2558415	.002617801
383	146689	56181887	19.5703858	7.2621675	.002610966
384	147456	56623104	19.5959179	7.2684824	.002604167
385	148225	57066625	19.6214169	7.2747864	.002597403
386	148996	57512456	19.6468827	7.2810794	.002590674
387	149769	57960603	19.6723156	7.2873617	.002583979
388	150544	58411072	19.6977156	7.2936330	.002577320
389	151321	58863869	19.7230829	7.2998936	.002570694
390	152100	59319000	19.7484177	7.3061436	.002564103
391	152881	59776471	19.7737199	7.3123828	.002557545
392	153664	60236288	19.7989899	7.3186114	.002551020
393	154449	60698457	19.8242276	7.3248295	.002544529
394	155236	61162984	19.8494332	7.3310369	.002538071
395	156025	61629875	19.8746069	7.3372339	.002531646
396	156816	62099136	19.8997487	7.3434205	.002525253
397	157609	62570773	19.9248588	7.3495966	.002518892
398	158404	63044792	19.9499373	7.3557624	.002512563
399	159201	63521199	19.9749844	7.3619178	.002506266
400	160000	64000000	20.0000000	7.3680630	.002500000
401	160801	64481201	20.0249844	7.3741979	.002493766
402	161604	64964808	20.0499377	7.3803227	.002487562
403	162409	65450827	20.0748599	7.3864373	.002481390
404	163216	65939264	20.0997512	7.3925418	.002475248
405	164025	66430125	20.1246118	7.3986363	.002469136
406	164836	66923416	20.1494417	7.4047206	.002463054
407	165649	67419143	20.1742410	7.4107950	.002457002
408	166464	67917312	20.1990099	7.4168595	.002450980
409	167281	68417929	20.2237484	7.4229142	.002444988
410	168100	68921000	20.2484567	7.4289589	.002439024
411	168921	69426531	20.2731349	7.4349938	.002433090
412	169744	69934528	20.2977831	7.4410189	.002427184
413	170569	70444997	20.3224014	7.4470342	.002421308
414	171396	70957944	20.3469899	7.4530399	.002415459
415	172225	71473375	20.3715488	7.4590359	.002409639
416	173056	71991296	20.3960781	7.4650223	.002403846
417	173889	72511713	20.4205779	7.4709991	.002398082
418	174724	73034632	20.4450483	7.4769664	.002392344
419	175561	73560059	20.4694895	7.4829242	.002386635
420	176400	74088000	20.4939015	7.4888724	.002380952
421	177241	74618461	20.5182845	7.4948113	.002375297
422	178084	75151448	20.5426386	7.5007406	.002369668
423	178929	75686967	20.5669638	7.5066607	.002364066
424	179776	76225024	20.5912603	7.5125715	.002358491
425	180625	76765625	20.6155281	7.5184730	.002352941
426	181476	77308776	20.6397674	7.5243652	.002347418
427	182329	77854483	20.6639783	7.5302482	.002341920
428	183184	78402752	20.6881609	7.5361221	.002336449
429	184041	78953589	20.7123152	7.5419867	.002331002
430	184900	79507000	20.7364414	7.5478423	.002325581
431	185761	80062991	20.7605395	7.5536888	.002320186
432	186624	80621568	20.7846097	7.5595263	.002314815
433	187489	81182737	20.8086520	7.5653548	.002309469
434	188356	81746504	20.8326667	7.5711743	.002304147

No.	Squares	Cubes	Square roots	Cube roots	Reciproc
435	189225	82312875	20.8566536	7.5769849	.002298
436	190096	82881856	20.8806130	7.5827865	.002293
437	190969	83453453	20.9045450	7.5885793	.002288
438	191844	84027672	20.9284495	7.5943633	.002283
439	192721	84604519	20.9523268	7.6001385	.002277
440	193600	85184000	20.9761770	7.6059049	.002272
441	194481	85766121	21.0000000	7.6116626	.002267
442	195364	86350888	21.0237960	7.6174116	.002262
443	196249	86938307	21.0475652	7.6231519	.002257
444	197136	87528384	21.0713075	7.6288837	.002252
445	198025	88121125	21.0950231	7.6346067	.002247
446	198916	88716536	21.1187121	7.6403213	.002242
447	199809	89314623	21.1423745	7.6460272	.002237
448	200704	89915392	21.1660105	7.6517247	.002232
449	201601	90518849	21.1896201	7.6574138	.002227
450	202500	91125000	21.2132034	7.6630943	.002222
451	203401	91733851	21.2367606	7.6687665	.002217
452	204304	92345408	21.2602916	7.6744303	.002212
453	205209	92959677	21.2837967	7.6800857	.00220
454	206116	93576664	21.3072758	7.6857328	.00220
455	207025	94196375	21.3307290	7.6913717	.00219
456	207936	94818816	21.3541565	7.6970023	.00219
457	208849	95443993	21.3775583	7.7026246	.00218
458	209764	96071912	21.4009346	7.7082388	.00218
459	210681	96702579	21.4242853	7.7138448	.00217
460	211600	97336000	21.4476106	7.7194426	.00217
461	212521	97972181	21.4709106	7.7250325	.00216
462	213444	98611128	21.4941853	7.7306141	.00216
463	214369	99252847	21.5174348	7.7361877	.00215
464	215296	99897344	21.5406592	7.7417532	.00215
465	216225	100544625	21.5638587	7.7473109	.00215
466	217156	101194696	21.5870331	7.7528606	.00214
467	218089	101847563	21.6101828	7.7584023	.00214
468	219024	102503232	21.6333077	7.7639361	.00213
469	219961	103161709	21.6564078	7.7694620	.00213
470	220900	103823000	21.6794834	7.7749801	.00212
471	221841	104487111	21.7025344	7.7804904	.00212
472	222784	105154048	21.7255610	7.7859928	.00211
473	223729	105823817	21.7485632	7.7914875	.00211
474	224676	106496424	21.7715411	7.7969745	.00210
475	225625	107171875	21.7944947	7.8024538	.00210
476	226576	107850176	21.8174242	7.8079254	.00210
477	227529	108531333	21.8403297	7.8133892	.00209
478	228484	109215352	21.8632111	7.8188456	.00209
479	229441	109902239	21.8860686	7.8242942	.00208
480	230400	110592000	21.9089023	7.8297353	.00208
481	231361	111284641	21.9317122	7.8351688	.00207
482	232324	111980168	21.9544984	7.8405949	.00207
483	233289	112678587	21.9772610	5.8514244	.00207
484	234256	113379904	22.0000000	7.8568281	.00206
485	235225	114084125	22.0227155	7.8622242	.00206
486	236196	114791256	22.0454077	7.8676130	.00205
487	237169	115501303	22.0680765	7.8729944	.0020
488	238144	116214272	22.0907220	7.8783684	.0020
489	239121	116930169	22.1133444	7.8837352	.0020
490	240100	117649000	22.1359436	7.8890946	.0020
491	241081	118370771	22.1585198	7.8944468	.0020
492	242064	119095488	22.1810730	7.8997917	.0020
493	243049	119823157	22.2036033	7.9051294	.0020
494	244036	120553784	22.2261108	7.9104599	.0020
495	245025	121287375	22.2485955	7.9157832	.0020
496	246016	122023936	22.2710575		

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
497	247009	122763473	22.2934968	7.9210994	.002012072
498	248004	123505992	22.3159136	7.9264085	.002008032
499	249001	124251499	22.3383079	7.9317104	.002004008
500	250000	125000000	22.3606798	7.9370053	.002000000
501	251001	125751501	22.3830293	7.9422931	.001996008
502	252004	126506008	22.4053565	7.9475739	.001992032
503	253009	127263527	22.4276615	7.9528477	.001988072
504	254016	128024064	22.4499443	7.9581144	.001984127
505	255025	128787625	22.4722051	7.9633743	.001980198
506	256036	129554216	22.4944438	7.9686271	.001976285
507	257049	130323843	22.5166605	7.9738731	.001972387
508	258064	131096512	22.5388553	7.9791122	.001968504
509	259081	131872229	22.5610283	7.9843444	.001964637
510	260100	132651000	22.5831796	7.9895697	.001960784
511	261121	133432831	22.6053091	7.9947883	.001956947
512	262144	134217728	22.6274170	8.0000000	.001953125
513	263169	135005697	22.6495033	8.0052049	.001949318
514	264196	135796744	22.6715681	8.0104032	.001945525
515	265225	136590875	22.6936114	8.0155946	.001941748
516	266256	137388096	22.7156334	8.0207794	.001937984
517	267289	138188413	22.7376340	8.0259574	.001934236
518	268324	138991832	22.7596134	8.0311287	.001930502
519	269361	139798359	22.7815715	8.0362935	.001926782
520	270400	140608000	22.8035085	8.0414515	.001923077
521	271441	141420761	22.8254244	8.0466030	.001919386
522	272484	142236648	22.8473193	8.0517479	.001915709
523	273529	143055667	22.8691933	8.0568862	.001912046
524	274576	143877824	22.8910463	8.0620180	.001908397
525	275625	144703125	22.9128785	8.0671432	.001904762
526	276676	145531576	22.9346899	8.0722620	.001901141
527	277729	146363183	22.9564806	8.0773743	.001897533
528	278784	147197952	22.9782506	8.0824800	.001893939
529	279841	148035889	23.0000000	8.0875794	.001890359
530	280900	148877000	23.0217289	8.0926723	.001886792
531	281961	149721291	23.0434372	8.0977589	.001883239
532	283024	150568768	23.0651252	8.1028390	.001879699
533	284089	151419437	23.0867928	8.1079128	.001876173
534	285156	152273304	23.1084400	8.1129803	.001872659
535	286225	153130375	23.1300670	8.1180414	.001869159
536	287296	153990656	23.1516738	8.1230962	.001865672
537	288369	154854153	23.1732605	8.1281447	.001862197
538	289444	155720872	23.1948270	8.1331870	.001858736
539	290521	156590819	23.2163735	8.1382230	.001855288
540	291600	157464000	23.2379001	8.1432529	.001851852
541	292681	158340421	23.2594067	8.1482765	.001848429
542	293764	159220088	23.2808935	8.1532939	.001845018
543	294849	160103007	23.3023604	8.1583051	.001841621
544	295936	160989184	23.3238076	8.1633102	.001838235
545	297025	161878625	23.3452351	8.1683092	.001834862
546	298116	162771336	23.3666429	8.1733020	.001831502
547	299209	163667323	23.3880311	8.1782888	.001828154
548	300304	164566592	23.4093998	8.1832695	.001824818
549	301401	165469149	23.4307490	8.1882441	.001821494
550	302500	166375000	23.4520788	8.1932127	.001818182
551	303601	167284151	23.4733892	8.1981753	.001814882
552	304704	168196608	23.4946802	8.2031319	.001811594
553	305809	169112377	23.5159520	8.2080825	.001808318
554	306916	170031464	23.5372046	8.2130271	.001805054
555	308025	170953875	23.5584380	8.2179657	.001801802
556	309136	171879616	23.5796522	8.2228985	.001798561
557	310249	172808693	23.6008474	8.2278254	.001795332

559	312481	174676879	23.6431808	8.2376614	.00178891
560	313600	175616000	23.6643191	8.2425706	.00178571
561	314721	176558481	23.6854386	8.2474740	.00178253
562	315844	177504328	23.7065392	8.2523715	.00177936
563	316969	178453547	23.7276210	8.2572633	.00177620
564	318096	179406144	23.7486842	8.2621492	.00177305
565	319225	180362125	23.7697286	8.2670294	.00176991
566	320356	181321496	23.7907545	8.2719039	.00176678
567	321489	182284263	23.8117618	8.2767726	.00176367
568	322624	183250432	23.8327506	8.2816355	.00176056
569	323761	184220009	23.8537209	8.2864928	.00175747
570	324900	185193000	23.8746728	8.2913444	.00175439
571	326041	186169411	23.8956063	8.2961903	.00175131
572	327184	187149248	23.9165215	8.3010304	.00174825
573	328329	188132517	23.9374184	8.3058651	.00174520
574	329476	189119224	23.9582971	8.3106941	.00174216
575	330625	190109375	23.9791576	8.3155175	.00173913
576	331776	191102976	24.0000000	8.3203353	.00173611
577	332929	192100033	24.0208243	8.3251475	.00173310
578	334084	193100552	24.0416306	8.3299542	.00173010
579	335241	194104539	24.0624188	8.3347553	.00172712
580	336400	195112000	24.0831891	8.3395509	.00172414
581	337561	196122941	24.1039416	8.3443410	.00172117
582	338724	197137368	24.1246762	8.3491256	.00171821
583	339889	198155287	24.1453929	8.3539047	.00171527
584	341056	199176704	24.1660919	8.3586784	.00171233
585	342225	200201625	24.1867732	8.3634466	.00170940
586	343396	201230056	24.2074369	8.3682095	.00170648
587	344569	202262003	24.2280829	8.3729668	.00170358
588	345744	203297472	24.2487113	8.3777188	.00170068
589	346921	204336469	24.2693222	8.3824653	.00169779
590	348100	205379000	24.2899156	8.3872065	.00169492
591	349281	206425071	24.3104916	8.3919423	.00169205
592	350464	207474688	24.3310501	8.3966729	.00168919
593	351649	208527857	24.3515913	8.4013981	.00168634
594	352836	209584584	24.3721152	8.4061180	.00168350
595	354025	210644875	24.3926218	8.4108326	.00168067
596	355216	211708736	24.4131112	8.4155419	.00167785
597	356409	212776173	24.4335834	8.4202460	.00167504
598	357604	213847192	24.4540385	8.4249448	.00167224
599	358801	214921799	24.4744765	8.4296383	.00166945
600	360000	216000000	24.4948974	8.4343267	.00166667
601	361201	217081801	24.5153013	8.4390098	.00166389
602	362404	218167208	24.5356883	8.4436877	.00166113
603	363609	219256227	24.5560583	8.4483605	.00165837
604	364816	220348864	24.5764115	8.4530281	.00165563
605	366025	221445125	24.5967478	8.4576906	.00165289
606	367236	222545016	24.6170673	8.4623479	.00165017
607	368449	223648543	24.6373700	8.4670001	.00164745
608	369664	224755712	24.6576560	8.4716471	.00164474
609	370881	225866529	24.6779254	8.4762892	.00164204
610	372100	226981000	24.6981781	8.4809261	.00163934
611	373321	228099131	24.7184142	8.4855579	.00163666
612	374544	229220928	24.7386338	8.4901848	.00163399
613	375769	230346397	24.7588368	8.4948065	.00163132
614	376996	231475544	24.7790234	8.4994233	.00162866
615	378225	232608375	24.7991935	8.5040350	.00162602
616	379456	233744896	24.8193473	8.5086417	.00162338
617	380689	234885113	24.8394847	8.5132435	.00162075
618	381924	236029032	24.8596058	8.5178403	.00161812
619	383161	237176659	24.8797106	8.5224321	.00161551

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
621	[illegible]	[illegible]	24.[illegible]	8.[illegible]	[illegible]
622	[illegible]	[illegible]	24.[illegible]	8.5381700	[illegible]
623	[illegible]	[illegible]	24.[illegible]	8.[illegible]	[illegible]
624	[illegible]	[illegible]	24.[illegible]	8.5453173	[illegible]
625	[illegible]	244140625	25.[illegible]	8.[illegible]	[illegible]
626	[illegible]	[illegible]	25.[illegible]	8.5544372	[illegible]
627	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
628	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
629	[illegible]	[illegible]	25.[illegible]	8.[illegible]	.001589825
630	[illegible]	[illegible]	25.[illegible]	8.5726109	[illegible]
631	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
632	399424	[illegible]	25.[illegible]	8.[illegible]	[illegible]
633	400689	253636137	25.[illegible]	8.[illegible]	[illegible]
634	[illegible]	[illegible]	25.1793566	8.[illegible]	[illegible]
635	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
636	[illegible]	[illegible]	25.[illegible]	8.[illegible]	.001572327
637	[illegible]	[illegible]	25.[illegible]	8.[illegible]	.001569859
638	[illegible]	[illegible]	25.2586619	8.[illegible]	[illegible]
639	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
640	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
641	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
642	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
643	[illegible]	[illegible]	25.[illegible]	8.[illegible]	.001555210
644	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
645	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
646	417316	[illegible]	25.[illegible]	8.[illegible]	[illegible]
647	[illegible]	[illegible]	25.4361947	8.[illegible]	[illegible]
648	[illegible]	[illegible]	25.[illegible]	8.6534974	[illegible]
649	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
650	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
651	[illegible]	[illegible]	25.5147016	8.[illegible]	[illegible]
652	425104	277167808	25.[illegible]	8.[illegible]	[illegible]
653	[illegible]	[illegible]	25.5538647	8.[illegible]	[illegible]
654	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
655	[illegible]	281011375	25.[illegible]	8.[illegible]	[illegible]
656	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
657	[illegible]	[illegible]	25.[illegible]	8.[illegible]	.001522070
658	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
659	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
660	[illegible]	[illegible]	25.[illegible]	8.[illegible]	.001515152
661	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
662	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
663	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
664	[illegible]	[illegible]	25.[illegible]	8.7241414	[illegible]
665	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
666	[illegible]	[illegible]	25.[illegible]	8.7328918	[illegible]
667	444889	[illegible]	25.[illegible]	8.[illegible]	[illegible]
668	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
669	447561	[illegible]	25.[illegible]	8.[illegible]	[illegible]
670	[illegible]	[illegible]	25.8843582	8.7503401	[illegible]
671	[illegible]	302111711	25.9036677	8.[illegible]	[illegible]
672	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
673	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
674	[illegible]	[illegible]	25.[illegible]	8.7677192	[illegible]
675	[illegible]	[illegible]	25.[illegible]	8.[illegible]	[illegible]
676	[illegible]	[illegible]	26.[illegible]	8.[illegible]	[illegible]
677	[illegible]	[illegible]	26.0192237	8.[illegible]	[illegible]
678	[illegible]	[illegible]	26.[illegible]	8.[illegible]	[illegible]
679	461041	[illegible]	26.0576284	8.[illegible]	[illegible]
680	[illegible]	[illegible]	26.[illegible]	8.[illegible]	[illegible]
681	463761	[illegible]	26.[illegible]	8.[illegible]	.001468429
682	465124	317214568	26.1151297	8.[illegible]	.001466276

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
683	466489	318611987	26.1342687	8.8065722	.001464129
684	467856	320013504	26.1533937	8.8108681	.001461988
685	469225	321419125	26.1725047	8.8151598	.001459854
686	470596	322828856	26.1916017	8.8194474	.001457726
687	471969	324242703	26.2106848	8.8237307	.001455604
688	473344	325660672	26.2297541	8.8280099	.001453488
689	474721	327082769	26.2488095	8.8322850	.001451379
690	476100	328509000	26.2678511	8.8365559	.001449275
691	477481	329939371	26.2868789	8.8408227	.001447178
692	478864	331373888	26.3058929	8.8450854	.001445087
693	480249	332812557	26.3248932	8.8493440	.001443001
694	481636	334255384	26.3438797	8.8535985	.001440922
695	483025	335702375	26.3628527	8.8578489	.001438849
696	484416	337153536	26.3818119	8.8620952	.001436782
697	485809	338608873	26.4007576	8.8663375	.001434720
698	487204	340068392	26.4196896	8.8705757	.001432665
699	488601	341532099	26.4386081	8.8748099	.001430615
700	490000	343000000	26.4575131	8.8790400	.001428571
701	491401	344472101	26.4764046	8.8832661	.001426534
702	492804	345948408	26.4952826	8.8874882	.001424501
703	494209	347428927	26.5141472	8.8917063	.001422475
704	495616	348913664	26.5329983	8.8959204	.001420455
705	497025	350402625	26.5518361	8.9001304	.001418440
706	498436	351895816	26.5706605	8.9043366	.001416431
707	499849	353393243	26.5894716	8.9085387	.001414427
708	501264	354894912	26.6082694	8.9127369	.001412429
709	502681	356400829	26.6270539	8.9169311	.001410437
710	504100	357911000	26.6458252	8.9211214	.001408451
711	505521	359425431	26.6645833	8.9253078	.001406470
712	506944	360944128	26.6833281	8.9294902	.001404494
713	508369	362467097	26.7020598	8.9336687	.001402525
714	509796	363994344	26.7207784	8.9378433	.001400560
715	511225	365525875	26.7394839	8.9420140	.001398601
716	512656	367061696	26.7581763	8.9461809	.001396648
717	514089	368601813	26.7768557	8.9503438	.001394700
718	515524	370146232	26.7955220	8.9545029	.001392758
719	516961	371694959	26.8141754	8.9586581	.001390821
720	518400	373248000	26.8328157	8.9628095	.001388889
721	519841	374805361	26.8514432	8.9669570	.001386963
722	521284	376367048	26.8700577	8.9711007	.001385042
723	522729	377933067	26.8886593	8.9752406	.001383126
724	524176	379503424	26.9072481	8.9793766	.001381215
725	525625	381078125	26.9258240	8.9835089	.001379310
726	527076	382657176	26.9443872	8.9876373	.001377410
727	528529	384240583	26.9629375	8.9917620	.001375516
728	529984	385828352	26.9814751	8.9958829	.001373626
729	531441	387420489	27.0000000	9.0000000	.001371742
730	532900	389017000	27.0185122	9.0041134	.001369863
731	534361	390617891	27.0370117	9.0082229	.001367989
732	535824	392223168	27.0554985	9.0123288	.001366120
733	537289	393832837	27.0739727	9.0164309	.001364256
734	538756	395446904	27.0924344	9.0205293	.001362398
735	540225	397065375	27.1108834	9.0246239	.001360544
736	541696	398688256	27.1293199	9.0287149	.001358696
737	543169	400315553	27.1477439	9.0328021	.001356852
738	544644	401947272	27.1661554	9.0368857	.001355014
739	546121	403583419	27.1845544	9.0409655	.001353180
740	547600	405224000	27.2029410	9.0450419	.001351351
741	549081	406869021	27.2213152	9.0491142	.001349528
742	550564	408518488	27.2396769	9.0531831	.001347709
743	552049	410172407	27.2580263	9.0572482	.001345895

No.	Squares	Cubes	Square roots	Cube roots	Reciproca
745	555025	413493625	27.2946881	9.0653677	.00134228
746	556516	415160936	27.3130006	9.0694220	.00134048
747	558009	416832723	27.3313007	9.0734726	.00133869
748	559504	418508992	27.3495887	9.0775197	.00133689
749	561001	420189749	27.3678644	9.0815631	.00133511
750	562500	421875000	27.3861279	9.0856030	.00133333
751	564001	423564751	27.4043792	9.0896392	.00133156
752	565504	425259008	27.4226184	9.0936719	.00132979
753	567009	426957777	27.4408455	9.0977010	.00132802
754	568516	428661064	27.4590604	9.1017265	.00132626
755	570025	430368875	27.4772633	9.1057485	.00132450
756	571536	432081216	27.4954542	9.1097669	.00132275
757	573049	433798093	27.5136330	9.1137818	.00132100
758	574564	435519512	27.5317998	9.1177931	.00131926
759	576081	437245479	27.5499546	9.1218010	.00131752
760	577600	438976000	27.5680975	9.1258053	.00131579
761	579121	440711081	27.5862284	9.1298061	.00131406
762	580644	442450728	27.6043475	9.1338034	.00131234
763	582169	444194947	27.6224546	9.1377971	.00131061
764	583696	445943744	27.6405499	9.1417874	.00130890
765	585225	447697125	27.6586334	9.1457742	.00130719
766	586756	449455096	27.6767050	9.1497576	.00130548
767	588289	451217663	27.6947648	9.1537375	.00130378
768	589824	452984832	27.7128129	9.1577139	.00130208
769	591361	454756609	27.7308492	9.1616869	.00130039
770	592900	456533000	27.7488739	9.1656565	.00129870
771	594441	458314011	27.7668808	9.1696225	.00129701
772	595984	460099648	27.7848880	9.1735852	.00129533
773	597529	461889917	27.8028775	9.1775445	.00129366
774	599076	463684824	27.8208555	9.1815003	.00129199
775	600625	465484375	27.8388218	9.1854527	.00129032
776	602176	467288576	27.8567766	9.1894018	.00128866
777	603729	469097433	27.8747197	9.1933474	.00128700
778	605284	470910952	27.8926514	9.1972897	.00128534
779	606841	472729139	27.9105715	9.2012286	.00128369
780	608400	474552000	27.9284801	9.2051641	.00128205
781	609961	476379541	27.9463772	9.2090962	.00128041
782	611524	478211768	27.9642629	9.2130250	.00127877
783	613089	480048687	27.9821372	9.2169505	.00127713
784	614656	481890304	28.0000000	9.2208726	.00127551
785	616225	483736625	28.0178515	9.2247914	.00127388
786	617796	485587656	28.0356915	9.2287068	.00127226
787	619369	487443403	28.0535203	9.2326189	.00127064
788	620944	489303872	28.0713377	9.2365277	.00126903
789	622521	491169069	28.0891438	9.2404333	.00126742
790	624100	493039000	28.1069386	9.2443355	.00126582
791	625681	494913671	28.1247222	9.2482344	.00126422
792	627264	496793088	28.1424946	9.2521300	.00126262
793	628849	498677257	28.1602557	9.2560224	.00126103
794	630436	500566184	28.1780056	9.2599114	.00125944
795	632025	502459875	28.1957444	9.2637973	.00125786
796	633616	504358336	28.2134720	9.2676798	.00125628
797	635209	506261573	28.2311884	9.2715592	.00125470
798	636804	508169592	28.2488938	9.2754352	.00125313
799	638401	510082399	28.2665881	9.2793081	.00125156
800	640000	512000000	28.2842712	9.2831777	.00125000
801	641601	513922401	28.3019434	9.2870440	.00124843
802	643204	515849608	28.3196045	9.2909072	.00124688
803	644809	517781627	28.3372546	9.2947671	.00124533
804	646416	519718464	28.3548938	9.2986239	.00124378
805	648025	521660125	28.3725219	9.3024775	.00124223
806	649636	523606616	28.3901391	9.3063278	.00124069

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
807	651249	525557943	28.4077454	9.3101750	.001239157
808	652864	527514112	28.4253408	9.3140190	.001237624
809	654481	529475129	28.4429253	9.3178599	.001236094
810	656100	531441000	28.4604989	9.3216975	.001234568
811	657721	533411731	28.4780617	9.3255320	.001233046
812	659344	535387328	28.4956137	9.3293634	.001231527
813	660969	537367797	28.5131549	9.3331916	.001230012
814	662596	539353144	28.5306852	9.3370167	.001228501
815	664225	541343375	28.5482048	9.3408386	.001226994
816	665856	543338496	28.5657137	9.3446575	.001225490
817	667489	545338513	28.5832119	9.3484731	.001223990
818	669124	547343432	28.6006993	9.3522857	.001222494
819	670761	549353259	28.6181760	9.3560952	.001221001
820	672400	551368000	28.6356421	9.3599016	.001219512
821	674041	553387661	28.6530976	9.3637049	.001218027
822	675684	555412248	28.6705424	9.3675051	.001216545
823	677329	557441767	28.6879766	9.3713022	.001215067
824	678976	559476224	28.7054002	9.3750963	.001213592
825	680625	561515625	28.7228132	9.3788873	.001212121
826	682276	563559976	28.7402157	9.3826752	.001210654
827	683929	565609283	28.7576077	9.3864600	.001209190
828	685584	567663552	28.7749891	9.3902419	.001207729
829	687241	569722789	28.7923601	9.3940206	.001206273
830	688900	571787000	28.8097206	9.3977964	.001204819
831	690561	573856191	28.8270706	9.4015691	.001203369
832	692224	575930368	28.8444102	9.4053387	.001201923
833	693889	578009537	28.8617394	9.4091054	.001200480
834	695556	580093704	28.8790582	9.4128690	.001199041
835	697225	582182875	28.8963666	9.4166297	.001197605
836	698896	584277056	28.9136646	9.4203873	.001196172
837	700569	586376253	28.9309523	9.4241420	.001194743
838	702244	588480472	28.9482297	9.4278936	.001193317
839	703921	590589719	28.9654967	9.4316423	.001191895
840	705600	592704000	28.9827535	9.4353880	.001190476
841	707281	594823321	29.0000000	9.4391307	.001189061
842	708964	596947688	29.0172363	9.4428704	.001187648
843	710649	599077107	29.0344623	9.4466072	.001186240
844	712336	601211584	29.0516781	9.4503410	.001184834
845	714025	603351125	29.0688837	9.4540719	.001183432
846	715716	605495736	29.0860791	9.4577999	.001182033
847	717409	607645423	29.1032644	9.4615249	.001180638
848	719104	609800192	29.1204396	9.4652470	.001179245
849	720801	611960049	29.1376046	9.4689661	.001177856
850	722500	614125000	29.1547595	9.4726824	.001176471
851	724201	616295051	29.1719043	9.4763957	.001175088
852	725904	618470208	29.1890390	9.4801061	.001173709
853	727609	620650477	29.2061637	9.4838136	.001172333
854	729316	622835864	29.2232784	9.4875182	.001170960
855	731025	625026375	29.2403830	9.4912200	.001169591
856	732736	627222016	29.2574777	9.4949188	.001168224
857	734449	629422793	29.2745623	9.4986147	.001166861
858	736164	631628712	29.2916370	9.5023078	.001165501
859	737881	633839779	29.3087018	9.5059980	.001164144
860	739600	636056000	29.3257566	9.5096854	.001162791
861	741321	638277381	29.3428015	9.5133699	.001161440
862	743044	640503928	29.3598365	9.5170515	.001160093
863	744769	642735647	29.3768616	9.5207303	.001158749
864	746496	644972544	29.3938769	9.5244063	.001157407
865	748225	647214625	29.4108823	9.5280794	.001156069
866	749956	649461896	29.4278779	9.5317497	.001154734
867	751689	651714363	29.4448637	9.5354172	.001153403

Squares	Cubes	Square roots	Cube roots	Reciprocals
62001	15438249	15.7797338	6.2911946	.004016064
62500	15625000	15.8113883	6.2996053	.004000000
63001	15813251	15.8429795	6.3079935	.003984064
63504	16003008	15.8745079	6.3163596	.003968254
64009	16194277	15.9059737	6.3247035	.003952569
64516	16387064	15.9373775	6.3330256	.003937008
65025	16581375	15.9687194	6.3413257	.003921569
65536	16777216	16.0000000	6.3496042	.003906250
66049	16974593	16.0312195	6.3578611	.003891051
66564	17173512	16.0623784	6.3660968	.003875969
67081	17373979	16.0934769	6.3743111	.003861004
67600	17576000	16.1245155	6.3825043	.003846154
68121	17779581	16.1554944	6.3906765	.003831418
68644	17984728	16.1864141	6.3988279	.003816794
69169	18191447	16.2172747	6.4069585	.003802281
69696	18399744	16.2480768	6.4150687	.003787879
70225	18609625	16.2788206	6.4231583	.003773585
70756	18821096	16.3095064	6.4312276	.003759398
71289	19034163	16.3401346	6.4392767	.003745318
71824	19248832	16.3707055	6.4473057	.003731343
72361	19465109	16.4012195	6.4553148	.003717472
72900	19683000	16.4316767	6.4633041	.003703704
73441	19902511	16.4620776	6.4712736	.003690037
73984	20123648	16.4924225	6.4792236	.003676471
74529	20346417	16.5227116	6.4871541	.003663004
75076	20570824	16.5529454	6.4950653	.003649635
75625	20796875	16.5831240	6.5029572	.003636364
76176	21024576	16.6132477	6.5108300	.003623188
76729	21253933	16.6433170	6.5186839	.003610108
77284	21484952	16.6733320	6.5265189	.003597122
77841	21717639	16.7032931	6.5343351	.003584229
78400	21952000	16.7332005	6.5421326	.003571429
78961	22188041	16.7630546	6.5499116	.003558719
79524	22425768	16.7928553	6.5576722	.003546099
80089	22335187	16.8226038	6.5654144	.003533569
80656	22903304	16.8522995	6.5731385	.003521127
81225	23149125	16.8819430	6.5808443	.003508772
81796	23393656	16.9115345	6.5885323	.003496503
82369	23639903	16.9410743	6.5962023	.003484321
82944	23887872	16.9705627	6.6038545	.003472222
83521	24137569	17.0000000	6.6114890	.003460208
84100	24389000	17.0293864	6.6191060	.003448276
84681	24642171	17.0587221	6.6267054	.003430426
85264	24897088	17.0880075	6.6342874	.003424658
85849	25153757	17.1172428	6.6418522	.003412969
86436	25412184	17.1464282	6.6493998	.003401361
87025	25672375	17.1755640	6.6569302	.003389831
87616	25934336	17.2046505	6.6644437	.003378378
88209	26198073	17.2336879	6.6719403	.003367003
88804	26463592	17.2626765	6.6794200	.003355705
89401	26730899	17.2916165	6.6868831	.003344482
90000	27000000	17.3205081	6.6943295	.003333333
90601	27270901	17.3493516	6.7017593	.003322259
91204	27543608	17.3781472	6.7091729	.003311258
91809	27818127	17.4068952	6.7165700	.003300330
92416	28094464	17.4355958	6.7239508	.003289474
93025	28372625	17.4642492	6.7313155	.003278689
93636	28652616	17.4928557	6.7386641	.003267974
94249	28934443	17.5214155	6.7459967	.003257329
94864	29218112	17.5499288	6.7533134	.003246753
95481	29503629	17.5783958	6.7606143	.003236246
96100	29791000	17.6068169	6.7678995	.003225806

Squares, Cubes, Square Roots, Cube Roots an

311	96721	30080231	17.6351921	6.7781
312	97344	30371328	17.6635217	6.7824
313	97969	30664297	17.6918060	6.7896
314	98596	30959144	17.7200451	6.7968
315	99225	31255875	17.7482393	6.8040
316	99856	31554496	17.7763888	6.8112
317	100489	31855013	17.8044938	6.8184
318	101124	32157432	17.8325545	6.8256
319	101761	32461759	17.8605711	6.8327
320	102400	32768000	17.8885438	6.8399
321	103041	33076161	17.9164729	6.8470
322	103684	33386248	17.9443584	6.8541
323	104329	33698267	17.9722008	6.8612
324	104976	34012224	18.0000000	6.8682
325	105625	34328125	18.0277564	6.8753
326	106276	34645976	18.0554701	6.8823
327	106929	34965783	18.0831413	6.8894
328	107584	35287552	18.1107703	6.8964
329	108241	35611289	18.1383571	6.9034
330	108900	35937000	18.1659021	6.9104
331	109561	36264691	18.1934054	6.9173
332	110224	36594368	18.2208672	6.9243
333	110889	36926037	18.2482876	6.9313
334	111556	37259704	18.2756669	6.9382
335	112225	37595375	18.3030052	6.9451
336	112896	37933056	18.3303028	6.9520
337	113569	38272753	18.3575598	6.9589
338	114244	38614472	18.3847763	6.9658
339	114921	38958219	18.4119526	6.9726
340	115600	39304000	18.4390889	6.9795
341	116281	39651821	18.4661853	6.9863
342	116964	40001688	18.4932420	6.9931
343	117649	40353607	18.5202592	7.0000
344	118336	40707584	18.5472370	7.0067
345	119025	41063625	18.5741756	7.0135
346	119716	41421736	18.6010752	7.0203
347	120409	41781923	18.6279360	7.0271
348	121104	42144192	18.6547581	7.0338
349	121801	42508549	18.6815417	7.0405
350	122500	42875000	18.7082869	7.0472
351	123201	43243551	18.7349940	7.0540
352	123904		18.7616630	7.0606
353	124609		18.7882942	7.0673
354	125316		18.8148877	7.0740
355	126025		18.8414437	7.0806
356	126736		18.8679623	7.0873
357	127449		18.8944436	7.0939
358	128164		18.9208879	7.100
359	128881		18.9472953	7.107
360	129600		18.9736660	7.113
361	130321		19.0000000	7.120
362	131044		19.0262976	7.126
363	131769		19.0525589	7.133
364	132496		19.0787840	7.140
365	133225		19.1049732	7.146
366	133956		19.1311265	7.153
367	134689		19.1572441	7.159
368	135424		19.1833261	7.166
369	136161		19.2093727	7.172
370	136900	50653000	19.2353841	7.179
371	137641	51064811	19.2613603	7.185
		51478848	19.2873015	7.191

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
621	385641	239483061	24.9198716	8.5316009	.00161030
622	386884	240641848	24.9399278	8.5361780	.00160771
623	388129	241804367	24.9599679	8.5407501	.00160512
624	389376	242970624	24.9799920	8.5453173	.00160256
625	390625	244140625	25.0000000	8.5498797	.00160000
626	391876	245314376	25.0199920	8.5544372	.00159744
627	393129	246491883	25.0399681	8.5589899	.00159489
628	394384	247673152	25.0599282	8.5635377	.00159235
629	395641	248858189	25.0798724	8.5680807	.00158982
630	396900	250047000	25.0998008	8.5726189	.00158730
631	398161	251239591	25.1197134	8.5771523	.00158478
632	399424	252435968	25.1396102	8.5816809	.00158227
633	400689	253636137	25.1594913	8.5862047	.00157977
634	401956	254840104	25.1793566	8.5907238	.00157728
635	403225	256047875	25.1992063	8.5952380	.00157480
636	404496	257259456	25.2190404	8.5997476	.00157232
637	405769	258474853	25.2388589	8.6042525	.00156985
638	407044	259694072	25.2586619	8.6087526	.00156739
639	408321	260917119	25.2784493	8.6132480	.00156494
640	409600	262144000	25.2982213	8.6177388	.00156250
641	410881	263374721	25.3179778	8.6222248	.00156006
642	412164	264609288	25.3377189	8.6267063	.00155763
643	413449	265847707	25.3574447	8.6311830	.00155521
644	414736	267089984	25.3771551	8.6356551	.00155279
645	416025	268336125	25.3968502	8.6401226	.00155038
646	417316	269586136	25.4165301	8.6445855	.00154798
647	418609	270840023	25.4361947	8.6490437	.00154559
648	419904	272097792	25.4558441	8.6534974	.00154321
649	421201	273359449	25.4754784	8.6579465	.00154083
650	422500	274625000	25.4950976	8.6623911	.00153846
651	423801	275894451	25.5147016	8.6668310	.00153609
652	425104	277167808	25.5342907	8.6712665	.00153374
653	426409	278445077	25.5538647	8.6756974	.00153139
654	427716	279726264	25.5734237	8.6801237	.00152905
655	429025	281011375	25.5929678	8.6845456	.00152671
656	430336	282300416	25.6124969	8.6889630	.00152439
657	431649	283593393	25.6320112	8.6933759	.00152207
658	432964	284890312	25.6515107	8.6977843	.00151975
659	434281	286191179	25.6709953	8.7021882	.00151745
660	435600	287496000	25.6904652	8.7065877	.00151515
661	436921	288804781	25.7099203	8.7109827	.00151285
662	438244	290117528	25.7293607	8.7153734	.00151057
663	439569	291434247	25.7487864	8.7197596	.00150829
664	440896	292754944	25.7681975	8.7241414	.001506024
665	442225	294079625	25.7875939	8.7285187	.001503759
666	443556	295408296	25.8069758	8.7328918	.001501502
667	444889	296740963	25.8263431	8.7372604	.001499250
668	446224	298077632	25.8456960	8.7416246	.001497006
669	447561	299418309	25.8650343	8.7459846	.001494768
670	448900	300763000	25.8843582	8.7503401	.001492537
671	450241	302111711	25.9036677	8.7546913	.001490313
672	451584	303464448	25.9229628	8.7590383	.001488095
673	452929	304821217	25.9422435	8.7633809	.001485884
674	454276	306182024	25.9615100	8.7677192	.001483680
675	455625	307546875	25.9807621	8.7720532	.001481481
676	456976	308915776	26.0000000	8.7763830	.001479290
677	458329	310288733	26.0192237	8.7807084	.001477105
678	459684	311665752	26.0384331	8.7850296	.001474926
679	461041	313046839	26.0576284	8.7893466	.004472754
680	462400	314432000	26.0768096	8.7936593	.001470588
681	463761	315821241	26.0959767	8.7979679	.001468429
682	465124	317214568	26.1151297	8.8022721	.001466276

n	Squares	Cubes	Square roots	Cube roots	Reciprocals
[illegible]	466489	318611987	26.1342687	8.8065722	.001464129
[illegible]	467856	320013504	26.1533937	8.8108681	.001461988
[illegible]	469225	321419125	26.1725047	8.8151598	.001459854
[illegible]	470596	322828856	26.1916017	8.8194474	.001457726
[illegible]	471969	324242703	26.2106848	8.8237307	.001455604
[illegible]	473344	325660672	26.2297541	8.8280099	.001453488
[illegible]	474721	327082769	26.2488095	8.8322850	.001451379
[illegible]	476100	328509000	26.2678511	8.8365559	.001449275
[illegible]	477481	329939371	26.2868789	8.8408227	.001447178
[illegible]	478864	331373888	26.3058929	8.8450854	.001445087
[illegible]	480249	332812557	26.3248932	8.8493440	.001443001
[illegible]	481636	334255384	26.3438797	8.8535985	.001440922
[illegible]	483025	335702375	26.3628527	8.8578489	.001438849
[illegible]	484416	337153536	26.3818119	8.8620952	.001436782
[illegible]	485809	338608873	26.4007576	8.8663375	.001434720
[illegible]	487204	340068392	26.4196896	8.8705757	.001432665
[illegible]	488601	341532099	26.4386081	8.8748099	.001430615
[illegible]	490000	343000000	26.4575131	8.8790400	.001428571
[illegible]	491401	344472101	26.4764046	8.8832661	.001426534
[illegible]	492804	345948408	26.4952826	8.8874882	.001424501
[illegible]	494209	347428927	26.5141472	8.8917063	.001422475
[illegible]	495616	348913664	26.5329983	8.8959204	.001420455
[illegible]	497025	350402625	26.5518361	8.9001304	.001418440
[illegible]	498436	351895816	26.5706605	8.9043366	.001416431
[illegible]	499849	353393243	26.5894716	8.9085387	.001414427
[illegible]	501264	354894912	26.6082694	8.9127369	.001412429
[illegible]	502681	356400829	26.6270539	8.9169311	.001410437
710	504100	357911000	26.6458252	8.9211214	.001408451
711	505521	359425431	26.6645833	8.9253078	.001406470
712	506944	360944128	26.6833281	8.9294902	.001404494
713	508369	362467097	26.7020598	8.9336687	.001402525
714	509796	363994344	26.7207784	8.9378433	.001400560
715	511225	365525875	26.7394839	8.9420140	.001398601
716	512656	367061696	26.7581763	8.9461809	.001396648
717	514089	368601813	26.7768557	8.9503438	.001394700
718	515524	370146232	26.7955220	8.9545029	.001392758
719	516961	371694959	26.8141754	8.9586581	.001390821
720	518400	373248000	26.8328157	8.9628095	.001388889
721	519841	374805361	26.8514432	8.9669570	.001386963
722	521284	376367048	26.8700577	8.9711007	.001385042
723	522729	377933067	26.8886593	8.9752406	.001383126
724	524176	379503424	26.9072481	8.9793766	.001381215
725	525625	381078125	26.9258240	8.9835089	.001379310
726	527076	382657176	26.9443872	8.9876373	.001377410
727	528529	384240583	26.9629375	8.9917620	.001375516
728	529984	385828352	26.9814751	8.9958829	.001373626
729	531441	387420489	27.0000000	9.0000000	.001371742
730	532900	389017000	27.0185122	9.0041134	.001369863
731	534361	390617891	27.0370117	9.0082229	.001367989
732	535824	392223168	27.0554985	9.0123288	.001366120
733	537289	393832837	27.0739727	9.0164309	.001364256
734	538756	395446904	27.0924344	9.0205293	.001362398
735	540225	397065375	27.1108834	9.0246239	.001360544
736	541696	398688256	27.1293199	9.0287149	.001358696
737	543169	400315553	27.1477439	9.0328021	.001356852
738	544644	401947272	27.1661554	9.0368857	.001355014
739	546121	403583419	27.1845544	9.0409655	.001353180
740	547600	405224000	27.2029410	9.0450419	.001351351
741	549081	406869021	27.2213152	9.0491142	.001349528
742	550564	408518488	27.2396769	9.0531831	.001347709
743	552049	410172407	27.2580263	9.0572482	.001345895
744	553536	411830784	27.2763634	9.0613098	.001344086

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
745	555025	413493625	27.2946881	9.0653677	.001342
746	556516	415160936	27.3130006	9.0694220	.001340
747	558009	416832723	27.3313007	9.0734726	.001338
748	559504	418508992	27.3495887	9.0775197	.001336
749	561001	420189749	27.3678644	9.0815631	.001335
750	562500	421875000	27.3861279	9.0856030	.001333
751	564001	423564751	27.4043792	9.0896392	.001331
752	565504	425259008	27.4226184	9.0936719	.001329
753	567009	426957777	27.4408455	9.0977010	.001328
754	568516	428661064	27.4590604	9.1017265	.001326
755	570025	430368875	27.4772633	9.1057485	.001324
756	571536	432081216	27.4954542	9.1097669	.001322
757	573049	433798093	27.5136330	9.1137818	.001321
758	574564	435519512	27.5317998	9.1177931	.001319
759	576081	437245479	27.5499546	9.1218010	.001317
760	577600	438976000	27.5680975	9.1258053	.001315
761	579121	440711081	27.5862284	9.1298061	.001314
762	580644	442450728	27.6043475	9.1338034	.001312
763	582169	444194947	27.6224546	9.1377971	.001310
764	583696	445943744	27.6405499	9.1417874	.001308
765	585225	447697125	27.6586334	9.1457742	.001307
766	586756	449455096	27.6767050	9.1497576	.001305
767	588289	451217663	27.6947648	9.1537375	.001303
768	589824	452984832	27.7128129	9.1577139	.001302
769	591361	454756609	27.7308492	9.1616869	.001300
770	592900	456533000	27.7488739	9.1656565	.001298
771	594441	458314011	27.7668808	9.1696225	.001297
772	595984	460099648	27.7848880	9.1735852	.001295
773	597529	461889917	27.8028775	9.1775445	.001293
774	599076	463684824	27.8208555	9.1815003	.001291
775	600625	465484375	27.8388218	9.1854527	.001290
776	602176	467288576	27.8567766	9.1894018	.001288
777	603729	469097433	27.8747197	9.1933474	.001287
778	605284	470910952	27.8926514	9.1972897	.001285
779	606841	472729139	27.9105715	9.2012286	.001283
780	608400	474552000	27.9284801	9.2051641	.001282
781	609961	476379541	27.9463772	9.2090962	.001280
782	611524	478211768	27.9642629	9.2130250	.001278
783	613089	480048687	27.9821372	9.2169505	.001277
784	614656	481890304	28.0000000	9.2208726	.001275
785	616225	483736625	28.0178515	9.2247914	.001273
786	617796	485587656	28.0356915	9.2287068	.001272
787	619369	487443403	28.0535203	9.2326189	.001270
788	620944	489303872	28.0713377	9.2365277	.001269
789	622521	491169069	28.0891438	9.2404333	.001267
790	624100	493039000	28.1069386	9.2443355	.001265
791	625681	494913671	28.1247222	9.2482344	.001264
792	627264	496793088	28.1424946	9.2521300	.001262
793	628849	498677257	28.1602557	9.2560224	.001261
794	630436	500566184	28.1780056	9.2599114	.001259
795	632025	502459875	28.1957444	9.2637973	.001257
796	633616	504358336	28.2134720	9.2676798	.001256
797	635209	506261573	28.2311884	9.2715592	.001254
798	636804	508169592	28.2488938	9.2754352	.001253
799	638401	510082399	28.2665881	9.2793081	.001251
800	640000	512000000	28.2842712	9.2831777	.001250
801	641601	513922401	28.3019434	9.2870440	.001248
802	643204	515849608	28.3196045	9.2909072	.001246
803	644809	517781627	28.3372546	9.2947671	.001245
804	646416	519718464	28.3548938	9.2986239	.001243
805	648025	521660125	28.3725219	9.3024775	.001242
806	649636	523606616	28.3901391	9.3063278	.001240

…uares, Cubes, Square Roots, Cube Roots and Reciprocals

Squares	Cubes	Square roots	Cube roots	Reciprocals
651249	525557943	28.4077454	9.3101750	.001239157
652864	527514112	28.4253408	9.3140190	.001237624
654481	529475129	28.4429253	9.3178599	.001236094
656100	531441000	28.4604989	9.3216975	.001234568
657721	533411731	28.4780617	9.3255320	.001233046
659344	535387328	28.4956137	9.3293634	.001231527
660969	537367797	28.5131549	9.3331916	.001230012
662596	539353144	28.5306852	9.3370167	.001228501
664225	541343375	28.5482048	9.3408386	.001226994
665856	543338496	28.5657137	9.3446575	.001225490
667489	545338513	28.5832119	9.3484731	.001223990
669124	547343432	28.6006993	9.3522857	.001222494
670761	549353259	28.6181760	9.3560952	.001221001
672400	551368000	28.6356421	9.3599016	.001219512
674041	553387661	28.6530976	9.3637049	.001218027
675684	555412248	28.6705424	9.3675051	.001216545
677329	557441767	28.6879766	9.3713022	.001215067
678976	559476224	28.7054002	9.3750963	.001213592
680625	561515625	28.7228132	9.3788873	.001212121
682276	563559976	28.7402157	9.3826752	.001210654
683929	565609283	28.7576077	9.3864600	.001209190
685584	567663552	28.7749891	9.3902419	.001207729
687241	569722789	28.7923601	9.3940206	.001206273
688900	571787000	28.8097206	9.3977964	.001204819
690561	573856191	28.8270706	9.4015691	.001203369
692224	575930368	28.8444102	9.4053387	.001201923
693889	578009537	28.8617394	9.4091054	.001200480
695556	580093704	28.8790582	9.4128690	.001199041
697225	582182875	28.8963666	9.4166297	.001197605
698896	584277056	28.9136646	9.4203873	.001196172
700569	586376263	28.9309523	9.4241420	.001194743
702244	588480472	28.9482297	9.4278936	.001193317
703921	590589719	28.9654967	9.4316423	.001191895
705600	592704000	28.9827535	9.4353880	.001190476
707281	594823321	29.0000000	9.4391307	.001189061
708964	596947688	29.0172363	9.4428704	.001187648
710649	599077107	29.0344623	9.4466072	.001186240
712336	601211584	29.0516781	9.4503410	.001184834
714025	603351125	29.0688837	9.4540719	.001183432
715716	605495736	29.0860791	9.4577999	.001182033
717409	607645423	29.1032644	9.4615249	.001180638
719104	609800192	29.1204396	9.4652470	.001179245
720801	611960049	29.1376046	9.4689661	.001177856
722500	614125000	29.1547595	9.4726824	.001176471
724201	616295051	29.1719043	9.4763957	.001175088
725904	618470208	29.1890390	9.4801061	.001173709
727609	620650477	29.2061637	9.4838136	.001172333
729316	622835864	29.2232784	9.4875182	.001170960
731025	625026375	29.2403830	9.4912200	.001169591
732736	627222016	29.2574777	9.4949188	.001168224
734449	629422793	29.2745623	9.4986147	.001166861
736164	631628712	29.2916370	9.5023078	.001165501
737881	633839779	29.3087018	9.5059980	.001164144
739600	636056000	29.3257566	9.5096854	.001162791
741321	638277381	29.3428015	9.5133699	.001161440
743044	640503928	29.3598365	9.5170515	.001160093
744769	642735647	29.3768616	9.5207303	.001158749
746496	644972544	29.3938769	9.5244063	.001157407
748225	647214625	29.4108823	9.5280794	.001156069
749956	649461896	29.4278779	9.5317497	.001154734
751689	651714363	29.4448637	9.5354172	.001153403
753424	653972032	29.4618397	9.5390818	.001152074

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
373	139129	51895117	10.3132070	7.1984040	[illegible]
374	139870	[illegible]13624	10.3390706	7.2048322	.002673797
375	[illegible]	52734375	10.3649167	7.2112479	[illegible]
376	141376	53157376	10.3907194	7.2176522	[illegible]
377	142129	[illegible]	10.4164878	7.[illegible]	[illegible]
378	142884	54010152	10.4422221	7.2304268	[illegible]
379	143641	[illegible]	10.4679223	7.[illegible]	[illegible]
380	144400	[illegible]0	10.4935887	7.2431565	.002631579
381	145161	[illegible]1	10.5192213	7.2495045	.002624672
382	145924	[illegible]8	10.5448203	7.2558415	.002617801
383	146689	[illegible]7	10.5703858	7.2621675	.00261[illegible]
384	147456	[illegible]4	10.5959179	7.2684824	.002604167
385	148225	[illegible]5	10.6214169	7.2747864	.002597403
386	148996	[illegible]6	10.6468827	7.[illegible]	.002590674
387	149769	[illegible]	10.6723156	7.2873617	.002583979
388	150544	[illegible]	10.6977156	7.2936330	.002577320
389	151321	[illegible]	10.7230830	7.[illegible]	.002570694
390	152100	59319000	10.7484177	7.3061436	.002564103
391	152881	59776471	10.7737199	7.3123828	.002557545
392	153664	[illegible]	10.79[illegible]	7.3186114	.002551020
393	154449	[illegible]457	10.8242276	7.3248295	.002544529
394	155236	61162984	10.8494332	7.3310369	.002538071
395	156025	61629875	10.8746069	7.3372339	.002531646
396	156816	62099136	10.8997487	7.3434205	.002525253
397	157609	62570773	10.9248588	7.3495966	.0025[illegible]
398	158404	63044792	10.9499373	7.3557624	.002512563
399	159201	63521199	10.9749844	7.3619178	.002506266
400	160000	64000000	[illegible]	7.[illegible]	.002500000
401	160801	64481201	20.0249844	7.3741979	.002493766
402	161604	64964808	20.0499377	7.3803227	.002487562
403	162409	65450827	[illegible]	7.3864373	.002481390
404	[illegible]	65939264	20.0997512	7.3925418	.002475248
405	164025	66430125	20.1246118	7.3986363	.002469136
406	164836	66923416	20.1494417	7.4047206	.002463054
407	[illegible]	67419143	20.1742410	7.4107950	.002457002
408	166464	67917312	20.1990099	7.4168595	.002450980
409	167281	[illegible]	20.2237484	7.4229142	.0024449[illegible]
410	[illegible]	68921000	20.2484567	[illegible]	.002439024
411	[illegible]	69426531	20.2731349	7.4349938	.002433090
412	[illegible]	69934528	20.2977831	7.4410189	.002427184
413	[illegible]	70444997	20.3224014	7.4470342	.002421308
414	[illegible]	[illegible]	20.3469899	7.4530399	.002415459
415	[illegible]	71473375	20.3715488	7.4590359	.002409639
416	[illegible]	71991296	20.3960781	7.4650223	.002403846
417	[illegible]	72511713	[illegible]	7.[illegible]	.002398082
418	[illegible]	73034632	20.4450483	7.4769664	.002392344
419	[illegible]	73560059	20.4694895	7.4829242	.002386635
420	[illegible]	74088000	20.4939015	7.4888724	.002380952
421	[illegible]241	74618461	20.5182845	7.4948113	.002375297
422	[illegible]084	75151448	20.5426386	7.5007406	[illegible]
423	[illegible]929	75686967	[illegible]	7.5066607	.002364066
424	[illegible]776	76225024	[illegible]	7.5125715	.002358491
425	[illegible]625	76765625	20.6155281	7.5184730	.002352941
426	[illegible]476	77308776	20.6397674	7.5243652	.002347418
427	[illegible]	77854483	20.6639783	7.5302482	.002341920
428	[illegible]184	78402752	20.6881609	7.5361221	.002336449
429	184041	78953589	20.7123152	7.5419867	.002331002
430	184900	79507000	20.7364414	7.5478423	.002325581
431	185761	80062991	20.7605395	7.5536888	.002320186
432	186624	80621568	20.7846097	7.5595263	.002314815
433	187489	81182737	20.8086520	7.5653548	.002309469
434	188356	81746504	20.8326667	7.5711743	.002304147

.	Squares	Cubes	Square roots	Cube roots	Reciprocal
35	189225	82312875	20.8566536	7.5769840	.00229885
36	190096	82881856	20.8806130	7.5827865	.00229357
37	190969	83453453	20.9045450	7.5885793	.00228833
38	191844	84027672	20.9284495	7.5943633	.00228310
39	192721	84604519	20.9523268	7.6001385	.00227790
40	193600	85184000	20.9761770	7.6059049	.00227272
41	194481	85766121	21.0000000	7.6116626	.00226757
42	195364	86350888	21.0237960	7.6174116	.00226244
43	196249	86938307	21.0475652	7.6231519	.00225733
44	197136	87528384	21.0713075	7.6288837	.00225225
445	198025	88121125	21.0950231	7.6346067	.00224719
446	198916	88716536	21.1187121	7.6403213	.00224215
447	199809	89314623	21.1423745	7.6460272	.00223713
448	200704	89915392	21.1660105	7.6517247	.00223214
449	201601	90518849	21.1896201	7.6574138	.00222717
450	202500	91125000	21.2132034	7.6630943	.00222222
451	203401	91733851	21.2367606	7.6687665	.00221729
452	204304	92345408	21.2602916	7.6744303	.00221238
453	205209	92959677	21.2837967	7.6800857	.00220750
454	206116	93576664	21.3072758	7.6857328	.00220264
455	207025	94196375	21.3307290	7.6913717	.00219780
456	207936	94818816	21.3541565	7.6970023	.00219298
457	208849	95443993	21.3775583	7.7026246	.00218818
458	209764	96071912	21.4009346	7.7082388	.00218340
459	210681	96702579	21.4242853	7.7138448	.00217864
460	211600	97336000	21.4476106	7.7194426	.00217391
461	212521	97972181	21.4709106	7.7250325	.00216919
462	213444	98611128	21.4941853	7.7306141	.00216450
463	214369	99252847	21.5174348	7.7361877	.00215982
464	215296	99897344	21.5406592	7.7417532	.00215517
465	216225	100544625	21.5638587	7.7473109	.00215053
466	217156	101194696	21.5870331	7.7528606	.00214592
467	218089	101847563	21.6101828	7.7584023	.00214132
468	219024	102503232	21.6333077	7.7639361	.00213675
469	219961	103161709	21.6564078	7.7694620	.00213219
470	220900	103823000	21.6794834	7.7749801	.00212766
471	221841	104487111	21.7025344	7.7804904	.00212314
472	222784	105154048	21.7255610	7.7859928	.00211864
473	223729	105823817	21.7485632	7.7914875	.00211416
474	224676	106496424	21.7715411	7.7969745	.00210970
475	225625	107171875	21.7944947	7.8024538	.00210526
476	226576	107850176	21.8174242	7.8079254	.00210084
477	227529	108531333	21.8403297	7.8133892	.0020964
478	228484	109215352	21.8632111	7.8188456	.0020920
479	229441	109902239	21.8860686	7.8242942	.0020876
480	230400	110592000	21.9089023	7.8297353	.0020833
481	231361	111284641	21.9317122	7.8351688	.0020790
482	232324	111980168	21.9544984	7.8405949	.0020746
483	233289	112678587	21.9772610	7.8460134	.0020703
484	234256	113379904	22.0000000	5.8514244	.0020661
485	235225	114084125	22.0227155	7.8568281	.0020618
486	236196	114791256	22.0454077	7.8622242	.0020576
487	237169	115501303	22.0680765	7.8676130	.0020533
488	238144	116214272	22.0907220	7.8729944	.0020491
489	239121	116930169	22.1133444	7.8783684	.0020449
490	240100	117649000	22.1359436	7.8837352	.0020408
491	241081	118370771	22.1585198	7.8890946	.0020366
492	242064	119095488	22.1810730	7.8944468	.0020325
493	243049	119823157	22.2036033	7.8997917	.0020288
[illegible]	244036	120553784	22.2261108	7.9051294	.0020242
[illegible]	[illegible]	121287375	22.2485955	7.9104599	.0020202

[redacted]	Squares	Cubes	Square roots	Cube roots	Reciprocals
497	247009	122763473	22 2934968	7.9210994	.002012072
498	248004	123505992	22.3159136	7.9264085	.002008032
499	249001	1[redacted]	22.3383079	7.9317104	.002004008
500	250000	125000000	22.[redacted]	7.9370053	.002000000
501	251001	125751501	22.3830293	7 9422931	.001996008
502	252004	126506008	22.4053565	7.9475739	.001992032
503	253009	127263527	22.4276615	7.9528477	.001988072
504	254016	128024064	22.[redacted]	7.9581144	.001984127
505	255025	128787625	22 4722051	7.9633743	.001980198
506	256036	129554216	22.4944438	7.9686271	.001976285
507	257049	130323843	22.5166605	7.9738731	.001972387
508	258064	131096512	22.5388553	7.9791122	.001968504
509	259081	131872229	22.5610283	7.9843444	.[redacted]
510	260100	132651000	22.5831796	7.9895697	.001960784
511	261121	133432831	22.6053091	7.9947883	.001956947
512	262144	134217728	22.6274170	8 0000000	.001953125
513	263169	135005697	22.6495033	8 0052049	.001949318
514	264196	135796744	[redacted] 6715681	8 0104032	.001945525
515	265225	136590875	22.6936114	8.0155946	.001941748
516	266256	137388096	22 7156334	8 0207794	001937984
517	267289	138188413	22.7376340	8.0259574	.001934236
518	268324	138991832	22.7596134	8 0311287	.001930502
519	269361	139798359	22.7815715	8.0362935	.001926782

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
559	312481	174676879	23.6431808	8.2376614	.00178890
560	313600	175616000	23.6643191	8.2425706	.00178571
561	314721	176558481	23.6854386	8.2474740	.00178253
562	315844	177504328	23.7065392	8.2523715	.00177935
563	316969	178453547	23.7276210	8.2572633	.00177619
564	318096	179406144	23.7486842	8.2621492	.00177305
565	319225	180362125	23.7697286	8.2670294	.00176991
566	320356	181321496	23.7907545	8.2719039	.00176678
567	321489	182284263	23.8117618	8.2767726	.00176366
568	322624	183250432	23.8327506	8.2816355	.00176056
569	323761	184220000	23.8537209	8.2864928	.00175746
570	324900	185193000	23.8746728	8.2913444	.00175438
571	326041	186169411	23.8956063	8.2961903	.00175131
572	327184	187149248	23.9165215	8.3010304	.00174825
573	328329	188132517	23.9374184	8.3058651	.00174520
574	329476	189119224	23.9582971	8.3106941	.00174216
575	330625	190109375	23.9791576	8.3155175	.00173913
576	331776	191102976	24.0000000	8.3203353	.00173611
577	332929	192100033	24.0208243	8.3251475	.00173310
578	334084	193100552	24.0416306	8.3299542	.00173010
579	335241	194104539	24.0624188	8.3347553	.00172711
580	336400	195112000	24.0831891	8.3395509	.00172413
581	337561	196122941	24.1039416	8.3443410	.00172117
582	338724	197137368	24.1246762	8.3491256	.00171821
583	339889	198155287	24.1453929	8.3539047	.00171526
584	341056	199176704	24.1660919	8.3586784	.00171232
585	342225	200201625	24.1867732	8.3634466	.00170940
586	343396	201230056	24.2074369	8.3682095	.00170648
587	344569	202262003	24.2280829	8.3729668	.00170357
588	345744	203297472	24.2487113	8.3777188	.00170068
589	346921	204336469	24.2693222	8.3824653	.00169779
590	348100	205379000	24.2899156	8.3872065	.00169491
591	349281	206425071	24.3104916	8.3919423	.00169204
592	350464	207474688	24.3310501	8.3966729	.00168918
593	351649	208527857	24.3515913	8.4013981	.00168634
594	352836	209584584	24.3721152	8.4061180	.00168350
595	354025	210644875	24.3926218	8.4108326	.00168067
596	355216	211708736	24.4131112	8.4155419	.00167785
597	356409	212776173	24.4335834	8.4202460	.00167504
598	357604	213847192	24.4540385	8.4249448	.00167224
599	358801	214921799	24.4744765	8.4296383	.00166944
600	360000	216000000	24.4948974	8.4343267	.00166666
601	361201	217081801	24.5153013	8.4390098	.00166389
602	362404	218167208	24.5356883	8.4436877	.00166113
603	363609	219256227	24.5560583	8.4483605	.00165837
604	364816	220348864	24.5764115	8.4530281	.00165562
605	366025	221445125	24.5967478	8.4576906	.00165289
606	367236	222545016	24.6170673	8.4623479	.00165016
607	368449	223648543	24.6373700	8.4670001	.00164744
608	369664	224755712	24.6576560	8.4716471	.00164473
609	370881	225866529	24.6779254	8.4762892	.00164203
610	372100	226981000	24.6981781	8.4809261	.00163934
611	373321	228099131	24.7184142	8.4855579	.00163666
612	374544	229220928	24.7386338	8.4901848	.00163398
613	375769	230346397	24.7588368	8.4948065	.00163132
614	376996	231475544	24.7790234	8.4994233	.00162866
615	378225	232608375	24.7991935	8.5040350	.00162601
616	379456	233744896	24.8193473	8.5086417	.00162337
617	380689	234885113	24.8394847	8.5132435	.00162074
618	381924	236029032	24.8596058	8.5178403	.00161812
619	383161	237176659	24.8797106	8.5224321	.00161550

Squares	Cubes	Square roots	Cube roots	Recip
385641	239	24.9198716	8.5316000	.0016
386884	240	24.9399278	8.5361780	.0016
388129	241	24.9599679	8.[illegible]	.0016
389376	242	24.9799920	8.5453173	.0016
390625	244	25.0000000	8.5498797	.0016
391876	245	25.[illegible]	8.5544372	.0015
393129	246	25.0399681	8.5589899	.0015
394384	247	25.0599282	8.5635377	.0015
395641	248	25.07[illegible]	8.5680807	.0015
396900	250047000	25.0998008	8.5726189	.0015
398161	251239591	25.1197134	8.57[illegible]	.0015
399424	252435968	25.1396102	8.58	.0015
400689	253636137	25.1594913	8.58	.0015
401956	254840104	25.17[illegible]	8.59	.0015
403225	256047875	25.1992063	8.59	.0015
404496	257259456	25.2190404	8.60	.0015
405769	258474853	25.2388589	8.60	.0015
407044	259694072	25.2586619	8.60	.0015
[illegible]	260917119	25.2784493	8.61	.0015
409600	2	25.2982213	8.6177388	.0015
410881	2	25.3179778	8.6222248	.0015
412164	2	25.3377189	8.6267063	.0015
413449	2	25.3574447	8.6311830	.0015
414736	2	25.3771551	8.6356551	.0015
416025	2	25.3968502	8.6401226	.0015
417316	2	25.4165301	8.6445855	.0015
418609	2	25.[illegible]	8.[illegible]	.0015
419904	2	25.4558441	8.6534974	.0015
421201	2	25.4754784	8.6579465	.0015
[illegible]	[illegible]	25.[illegible]	8.6623911	.00153
[illegible]	275894451	25.5147016	8.6668310	.00153
425104	277167808	25.5342907	8.6712665	.00153
426409	278445077	25.5538647	8.6756974	.00153
427716	279726264	25.5734237	8.6801237	.00152
[illegible]	281011375	25.5929678	8.6845456	.00152
430336	[illegible]	25.6[illegible]	8.6889630	.00152
431649	283593393	25.6320112	8.6933759	.00152
432964	284890312	25.6515107	8.6977843	.00151
[illegible]	286191179	25.6709953	8.7[illegible]	.0015174
435600	287496000	25.6904652	8.7065877	.0015151
436921	288804781	25.7099203	8.7109827	.00151
438244	290117528	25.7293607	8.7153734	.00151
439569	291434247	25.7487864	8.7197596	.00150
440896	2 144	25.[illegible]	8.7241414	.00150
442225	2 125	25.7875939	8.7285187	.00150
443556	2 296	25.8069758	8.7328918	.0015015
444889	2 963	25.8263431	8.7[illegible]	.00149
446224	2 632	25.8456960	8.7416246	.00149
[illegible]	2 309	25.8650343	8.7459846	[illegible]
448900	[illegible]	25.8843582	8.7503401	.00149
450241	302111711	25.9036677	8.7546913	.00149
451584	303464448	25.9229628	8.[illegible]	.00148
452929	304821217	25.9422435	8.[illegible]	.00148
454276	306182024	25.9615100	8.7677192	[illegible]
455625	307546875	25.9807621	8.7720532	.0014814
456976	308915776	26.0000000	8.7763830	.00147
458329	310288733	26.0192237	8.7807084	.0014771
459684	311665752	26.0384331	8.7850296	.0014749
461041	313046839	26.0576284	8.7893466	.0014727
462400	314432000	26.0768096	8.7936593	.00147
463761	315821241	26.0959767	8.7979679	.00146
465124	317214568	26.1151297	8.8022721	.00146

Squares	Cubes	Square Roots	Cube Roots	Reciprocals
[illegible]	[illegible]	26 [illegible]	8 [illegible]	[illegible]
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[illegible]	[illegible]	27 0185122	9 [illegible]	[illegible]
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[illegible]	[illegible]	27 [illegible]	9 0613098	[illegible]

No.	Squares	Cubes	Square roots	Cube roots	Recipro
745	555025	413493625	27.2946881	9.0653677	.001342
746	556516	415160936	27.3130006	9.0694220	.001340
747	558009	416832723	27.3313007	9.0734726	.001338
748	559504	418508992	27.3495887	9.0775197	.001336
749	561001	420189749	27.3678644	9.0815631	.001335
750	562500	421875000	27.3861279	9.0856030	.001333
751	564001	423564751	27.4043792	9.0896392	.001331
752	565504	425259008	27.4226184	9.0936719	.001329
753	567009	426957777	27.4408455	9.0977010	.001328
754	568516	428661064	27.4590604	9.1017265	.001326
755	570025	430368875	27.4772633	9.1057485	.001324
756	571536	432081216	27.4954542	9.1097669	.001322
757	573049	433798093	27.5136330	9.1137818	.001321
758	574564	435519512	27.5317998	9.1177931	.001319
759	576081	437245479	27.5499546	9.1218010	.001317
760	577600	438976000	27.5680975	9.1258053	.001315
761	579121	440711081	27.5862284	9.1298061	.001314
762	580644	442450728	27.6043475	9.1338034	.001312
763	582169	444194947	27.6224546	9.1377971	.001310
764	583696	445943744	27.6405499	9.1417874	.001308
765	585225	447697125	27.6586334	9.1457742	.001307
766	586756	449455096	27.6767050	9.1497576	.001305
767	588289	451217663	27.6947648	9.1537375	.001303
768	589824	452984832	27.7128129	9.1577139	.001302
769	591361	454756609	27.7308492	9.1616869	.001300
770	592900	456533000	27.7488739	9.1656565	.001298
771	594441	458314011	27.7668808	9.1696225	.001297
772	595984	460099648	27.7848880	9.1735852	.001295
773	597529	461889917	27.8028775	9.1775445	.001293
774	599076	463684824	27.8208555	9.1815003	.001291
775	600625	465484375	27.8388218	9.1854527	.001290
776	602176	467288576	27.8567766	9.1894018	.001288
777	603729	469097433	27.8747197	9.1933474	.001287
778	605284	470910952	27.8926514	9.1972897	.001285
779	606841	472729139	27.9105715	9.2012286	.001283
780	608400	474552000	27.9284801	9.2051641	.001282
781	609961	476379541	27.9463772	9.2090962	.001280
782	611524	478211768	27.9642629	9.2130250	.001278
783	613089	480048687	27.9821372	9.2169505	.001277
784	614656	481890304	28.0000000	9.2208726	.001275
785	616225	483736625	28.0178515	9.2247914	.001273
786	617796	485587656	28.0356915	9.2287068	.001272
787	619369	487443403	28.0535203	9.2326189	.001270
788	620944	489303872	28.0713377	9.2365277	.001269
789	622521	491169069	28.0891438	9.2404333	.001267
790	624100	493039000	28.1069386	9.2443355	.001265
791	625681	494913671	28.1247222	9.2482344	.001264
792	627264	496793088	28.1424946	9.2521300	.001262
793	628849	498677257	28.1602557	9.2560224	.001261
794	630436	500566184	28.1780056	9.2599114	.001259
795	632025	502459875	28.1957444	9.2637973	.001257
796	633616	504358336	28.2134720	9.2676798	.001256
797	635209	506261573	28.2311884	9.2715592	.001254
798	636804	508169592	28.2488938	9.2754352	.001253
799	638401	510082399	28.2665881	9.2793081	.001251
800	640000	512000000	28.2842712	9.2831777	.001250
801	641601	513922401	28.3019434	9.2870440	.001248
802	643204	515849608	28.3196045	9.2909072	.001246
803	644809	517781627	28.3372546	9.2947671	.001245
804	646416	519718464	28.3548938	9.2986239	.001243
805	648025	521660125	28.3725219	9.3024775	.001242
806	649636	523606616	28.3901391	9.3063278	.001240

Squares	Cubes	Square roots	Cube roots	Reciprocals
651249	525557943	28.4077454	9.3101750	.001239157
652864	527514112	28.4253408	9.3140190	.001237624
654481	529475129	28.4429253	9.3178599	.001236094
656100	531441000	28.4604989	9.3216975	.001234568
657721	533411731	28.4780617	9.3255320	.001233046
659344	535387328	28.4956137	9.3293634	.001231527
660969	537367797	28.5131549	9.3331916	.001230012
662596	539353144	28.5306852	9.3370167	.001228501
664225	541343375	28.5482048	9.3408386	.001226994
665856	543338496	28.5657137	9.3446575	.001225490
667489	545338513	28.5832119	9.3484731	.001223990
669124	547343432	28.6006993	9.3522857	.001222494
670761	549353259	28.6181760	9.3560952	.001221001
672400	551368000	28.6356421	9.3599016	.001219512
674041	553387661	28.6530976	9.3637049	.001218027
675684	555412248	28.6705424	9.3675051	.001216545
677329	557441767	28.6879766	9.3713022	.001215067
678976	559476224	28.7054002	9.3750963	.001213592
680625	561515625	28.7228132	9.3788873	.001212121
682276	563559976	28.7402157	9.3826752	.001210654
683929	565609283	28.7576077	9.3864600	.001209190
685584	567663552	28.7749891	9.3902419	.001207729
687241	569722789	28.7923601	9.3940206	.001206273
688900	571787000	28.8097206	9.3977964	.001204819
690561	573856191	28.8270706	9.4015691	.001203369
692224	575930368	28.8444102	9.4053387	.001201923
693889	578009537	28.8617394	9.4091054	.001200480
695556	580093704	28.8790582	9.4128690	.001199041
697225	582182875	28.8963666	9.4166297	.001197605
698896	584277056	28.9136646	9.4203873	.001196172
700569	586376263	28.9309523	9.4241420	.001194743
702244	588480472	28.9482297	9.4278936	.001193317
703921	590589719	28.9654967	9.4316423	.001191895
705600	592704000	28.9827535	9.4353880	.001190476
707281	594823321	29.0000000	9.4391307	.001189061
708964	596947688	29.0172363	9.4428704	.001187648
710649	599077107	29.0344623	9.4466072	.001186240
712336	601211584	29.0516781	9.4503410	.001184834
714025	603351125	29.0688837	9.4540719	.001183432
715716	605495736	29.0860791	9.4577999	.001182033
717409	607645423	29.1032644	9.4615249	.001180638
719104	609800192	29.1204396	9.4652470	.001179245
720801	611960049	29.1376046	9.4689661	.001177856
722500	614125000	29.1547595	9.4726824	.001176471
724201	616295051	29.1719043	9.4763957	.001175088
725904	618470208	29.1890390	9.4801061	.001173709
727609	620650477	29.2061637	9.4838136	.001172333
729316	622835864	29.2232784	9.4875182	.001170960
731025	625026375	29.2403830	9.4912200	.001169591
732736	627222016	29.2574777	9.4949188	.001168224
734449	629422793	29.2745623	9.4986147	.001166861
736164	631628712	29.2916370	9.5023078	.001165501
737881	633839779	29.3087018	9.5059980	.001164144
739600	636056000	29.3257566	9.5096854	.001162791
741321	638277381	29.3428015	9.5133699	.001161440
743044	640503928	29.3598365	9.5170515	.001160093
744769	642735647	29.3768616	9.5207303	.001158749
746496	644972544	29.3938769	9.5244063	.001157407
748225	647214625	29.4108823	9.5280794	.001156069
749956	649461896	29.4278779	9.5317497	.001154734
751689	651714363	29.4448637	9.5354172	.001153403
753424	653972032	29.4618397	9.5390818	.001152074

No.	Squares	Cubes	Square roots	Cube roots	Reciprocals
[illegible]	755161	[illegible]	29 [illegible]	9 [illegible]	[illegible]
70	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
71	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
72	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
73	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
74	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
75	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
76	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
77	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
78	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
79	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
80	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
81	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
82	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
83	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
84	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
85	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
86	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
87	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
88	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
89	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
90	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
91	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
92	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
93	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
94	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
95	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
96	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
97	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
98	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
99	[illegible]	[illegible]	29 [illegible]	9 [illegible]	[illegible]
00	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
01	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
02	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
03	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
04	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
05	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
06	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
07	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
08	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
09	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]
[illegible]	[illegible]	[illegible]	30 [illegible]	9 [illegible]	[illegible]

Squares	Cubes	Square roots	Cube roots	Reciprocal
866761	806	30.	9.70	.001074
868624	809	30.	9.71	.001072
870489	812	30.	9.7	[illegible]
872356	814	30.	9.7	.001070
874225	817	30.	9.7	.00106
876096	820	30.	9.71	.00106
877969	822	30.	9.71	.001067
879844	825	30.	9.71	.001066
881721	827	30.	9.71	.001064
883600	8	30.656	9.7	.001063
885481	8	30.675	9.7	.001062
887364	8	30.692	9.8	.001061
889249	8	30.706	9.8	.001060
891136	8	30.724	9.8	.001059
893025	8	30.740	9.8131989	.001058
894916	8	30.757	9.8166591	.001057
896809	8	30.773	9.8201169	.001055
898704	8	[illegible]	9.8235723	.001054
900601	8	30.805	9.8270252	.001053
	857375000	30.82		.001052
	860085351	30.83		.001051
	862801408	30.85		.001050
	865523177	30.87		.00104931
	868250664	30.88		.00104821
	870983575	30.90		.00104712
	873722816	30.91		.00104602
	876467493	30.93		.00104493
	879217912	30.95		.00104384
[illegible]	881974079	30.9677251		.00104275
	8	30.9838668	9.8648483	.001041667
	8	31.0000000	9.8682724	.001040583
	8	31.0161248	9.8716941	.001039501
	8	31.0322413		.001038422
	8	31.0483494		.001037344
	[illegible]	31.0644491		.001036269
	[illegible]	31.0805405		.001035197
	9	31.0966236		.001034126
	9	31.1126984		.001033058
[illegible]	9	31.1287648		.001031992
940900	9	230	9.8989830	.001030928
942841	9	729	9.9023835	.001029866
944784	9	145	9.9057817	.001028807
946729	9	479	9.1	.00102774
948676	9	731	9.1	.00102669
950625	9	900	9.1	.00102564
952576	9	987	9.1	.00102459
954529	9	992	9.1	.00102354
956484	9	915	9.1	.00102249
958441	[illegible]	757	9.1	.00102145
960400	000	617		.00102040
962361	141	195		.00101936
964324	168	792		.00101833
966289	087	308		.00101729
968256	904	743		.00101626
970225	.625	[illegible]		.00101522
972196	256	369		.00101419
974169	803	561		.00101317
976144	272	673		.00101214
978121	1669	704		.0010111
980100	970299000	31.4642654	9.9665549	.0010101
982081	973242271	31.4801525	9.9699095	.0010090
984064	976191488	[illegible]	9.9732619	.0010080

No.	Squares	Cubes	Square roots	Cube roots	Recipr
993	986049	979146657	31.5119025	9.9766120	.00100
994	988036	982107784	31.5277655	9.9799599	.00100
995	990025	985074875	31.5436206	9.9833055	.00100
996	992016	988047936	31.5594677	9.9866488	.00100
997	994009	991026973	31.5753068	9.9899900	.00100
998	996004	994011992	31.5911380	9.9933289	.00100
999	998001	997002999	31.6069613	9.9966656	.00100
1000	1000000	1000000000	31.6227766	10.0000000	.00100
1001	1002001	1003003001	31.6385840	10.0033322	.00099
1002	1004004	1006012008	31.6543836	10.0066622	.00099
1003	1006009	1009027027	31.6701752	10.0099899	.00099
1004	1008016	1012048064	31.6859590	10.0133155	.00099
1005	1010025	1015075125	31.7017349	10.0166389	.00099
1006	1012036	1018108216	31.7175030	10.0199601	.00099
1007	1014049	1021147343	31.7332633	10.0232791	.00099
1008	1016064	1024192512	31.7490157	10.0265958	.00099
1009	1018081	1027243729	31.7647603	10.0299104	.00099
1010	1020100	1030301000	31.7804972	10.0332228	.00099
1011	1022121	1033364331	31.7962262	10.0365330	.00098
1012	1024144	1036433728	31.8119474	10.0398410	.00098
1013	1026169	1039509197	31.8276609	10.0431469	.00098
1014	1028196	1042590744	31.8433666	10.0464506	.00098
1015	1030225	1045678375	31.8590646	10.0497521	.00098
1016	1032256	1048772096	31.8747549	10.0530514	.00098
1017	1034289	1051871913	31.8904374	10.0563485	.00098
1018	1036324	1054977832	31.9061123	10.0596435	.00098
1019	1038361	1058089859	31.9217794	10.0629364	.00098
1020	1040400	1061208000	31.9374388	10.0662271	.00098
1021	1042441	1064332261	31.9530906	10.0695156	.00097
1022	1044484	1067462648	31.9687347	10.0728020	.00097
1023	1046529	1070599167	31.9843712	10.0760863	.00097
1024	1048576	1073741824	32.0000000	10.0793684	.00097
1025	1050625	1076890625	32.0156212	10.0826484	.00097
1026	1052676	1080045576	32.0312348	10.0859262	.00097
1027	1054729	1083206683	32.0468407	10.0892019	.00097
1028	1056784	1086373952	32.0624391	10.0924755	.00097
1029	1058841	1089547389	32.0780298	10.0957469	.00097
1030	1060900	1092727000	32.0936131	10.0990163	.00097
1031	1062961	1095912791	32.1091887	10.1022835	.00096
1032	1065024	1099104768	32.1247568	10.1055487	.00096
1033	1067089	1102302937	32.1403173	10.1088117	.00096
1034	1069156	1105507304	32.1558704	10.1120726	.00096
1035	1071225	1108717875	32.1714159	10.1153314	.00096
1036	1073296	1111934656	32.1869539	10.1185882	.00096
1037	1075369	1115157653	32.2024844	10.1218428	.00096
1038	1077444	1118386872	32.2180074	10.1250953	.00096
1039	1079521	1121622319	32.2335229	10.1283457	.00096
1040	1081600	1124864000	32.2490310	10.1315941	.00096
1041	1083681	1128111921	32.2645316	10.1348403	.00096
1042	1085764	1131366088	32.2800248	10.1380845	.00095
1043	1087849	1134626507	32.2955105	10.1413266	.00095
1044	1089936	1137893184	32.3109888	10.1445667	.00095
1045	1092025	1141166125	32.3264598	10.1478047	.00095
1046	1094116	1144445336	32.3419233	10.1510406	.00095
1047	1096209	1147730823	32.3573794	10.1542744	.00095
1048	1098304	1151022592	32.3728281	10.1575062	.00095
1049	1100401	1154320649	32.3882695	10.1607359	.00095
1050	1102500	1157625000	32.4037035	10.1639636	.00095
1051	1104601	1160935651	32.4191301	10.1671893	.000951
1052	1106704	1164252608	32.4345495	10.1704129	.000950
1053	1108809	1167575877	32.4499615	10.1736344	.000949
1054	1110916	1170905464	32.4653662	10.1768539	.000948

2. WEIGHTS AND MEASURES

Measures of Length

= 1 foot
= 1 yard = 36 inches
= 1 rod = 198 inches = 16½ feet
= 1 furlong = 7 920 inches = 660 feet = 220 yards
= 1 mile = 63 360 inches = 5 280 feet = 1 760 yards = 320 rods
= 0.0000158 of a mile

Gunter's Chain

7.92 inches = 1 link
100 links = 1 chain = 4 rods = 66 feet
80 chains = 1 mile

Ropes and Cables

6 feet = 1 fathom   120 fathoms = 1 cable's length

Table Showing Inches Expressed in Decimals of a Foot

0	1	2	3	4	5	6	7	8	9	10	11
Foot	.0833	.1667	.2500	[illegible]	.4167	.5000	.5833	.6667	.7500	.8333	.9167
.0026	.0859	.1693	.2526	.3359	.4193	.5026	.5859	.6693	.7526	.8359	.9193
.0052	.0885	.1719	.2552	.3385	.4219	.5052	.5885	.6719	.7552	.8385	.9219
.0078	.0911	.1745	.2578	.3411	.4245	.5078	.5911	.6745	.7578	.8411	.9245
.0104	.0938	.1771	.2604	.3438	.4271	.5104	.5938	.6771	.7604	.8438	.9271
.0130	.0964	.1797	.2630	.3464	.4297	.5130	.5964	.6797	.7630	.8464	.9297
.0156	.0990	.1823	.2656	.3490	.4323	.5156	.5990	.6823	.7656	.8490	.9323
.0182	.1016	.1849	.2682	.3516	.4349	.5182	.6016	.6849	.7682	.8516	.9349
.0208	.1042	.1875	.2708	.3542	.4375	.5208	[illegible]	.6875	.7708	.8542	.9375
.0234	.1068	.1901	.2734	.3568	.4401	.5234	.6068	.6901	.7734	.8568	.9401
.0260	.1094	.1927	.2760	.3594	.4427	.5260	.6094	.6927	.7760	.8594	.9427
.0286	.1120	.1953	.2786	.3620	.4453	.5286	.6120	.6953	.7786	.8620	.9453
.0313	.1146	.1979	.2813	.3646	.4479	.5313	.6146	.6979	.7813	.8646	.9479
.0339	.1172	.2005	[illegible]	.3672	.4505	.5339	.6172	[illegible]	.7839	.8672	.9505
.0365	.1198	.2031	.2865	.3698	.4531	.5365	.6198	.7031	.7865	.8698	.9531
.0391	.1224	.2057	.2891	.3724	.4557	.5391	.6224	.7057	.7891	.8724	.9557
.0417	.1250	.2083	.2917	[illegible]	.4583	.5417	.6250	.7083	.7917	.8750	.9583
.0443	.1276	.2109	.2943	[illegible]	[illegible]	.5443	.6276	.7109	.7943	.8776	.9609
.0469	.1302	.2135	.2969	[illegible]	.4635	.5469	.6302	.7135	.7969	.8802	.9635
.0495	.1328	.2161	.2995	[illegible]	.4661	.5495	.6328	[illegible]	.7995	.8828	.9661
.0521	.1354	.2188	.3021	[illegible]	.4688	.5521	.6354	[illegible]	.8021	.8854	.9688
.0547	.1380	.2214	.3047	[illegible]	.4714	.5547	.6380	.7214	[illegible]	.8880	.9714
.0573	.1406	.2240	.3073	[illegible]	.4740	[illegible]	.6406	.7240	.8073	.8906	.9740
.0599	.1432	.2266	.3099	[illegible]	.4766	.5599	.6432	.7266	.8099	.8932	.9766
.0625	.1458	.2292	.3125	[illegible]	.4792	.5625	.6458	.7292	.8125	.8958	.9792
.0651	.1484	.2318	.3151	[illegible]	[illegible]	.5651	.6484	.7318	.8151	[illegible]	.9818
.0677	.1510	.2344	.3177	[illegible]	[illegible]	.5677	.6510	.7344	.8177	.9010	.9844
.0703	.1536	.2370	.3203	[illegible]	.4870	.5703	.6536	.7370	.8203	[illegible]	.9870
.0729	.1563	.2396	.3229	[illegible]	.4896	.5729	.6563	.7396	.8229	.9063	.9896
.0755	.1589	.2422	.3255	[illegible]	.4922	[illegible]	.6589	.7422	.8255	.9089	[illegible]
.0781	.1615	.2448	.3281	[illegible]	.4948	.5781	.6615	.7448	.8281	.9115	.9948
.0807	.1641	.2474	.3307	[illegible]	.4974	[illegible]	.6641	.7474	.8307	.9141	.9974
0	1	2	3	4	5	6	7	8	9	10	11

Decimal Equivalents for Fractions of an Inch

1/32	1/64	Decimals	Fractions	1/32	1/64	Decimals	
......	1	0.015625	....	......	33	0.515625	..
1	2	0.03125	....	17	34	0.53125	..
......	3	0.046875	....	......	35	0.546875	..
2	4	0.0625	1/16	18	36	0.5625	
......	5	0.078125	....	......	37	0.578125	..
3	6	0.09375	....	19	38	0.59375	..
......	7	0.109375	....	......	39	0.609375	..
4	8	0.125	1/8	20	40	0.625	
......	9	0.140625	....	......	41	0.640625	..
5	10	0.15625	....	21	42	0.65625	..
......	11	0.171875	....	......	43	0.671875	...
6	12	0.1875	3/16	22	44	0.6875	
......	13	0.203125	....	......	45	0.703125	...
7	14	0.21875	....	23	46	0.71875	...
......	15	0.234375	....	......	47	0.734375	...
8	16	0.25	1/4	24	48	0.75	
......	17	0.265625	....	......	49	0.765625	...
9	18	0.28125	....	25	50	0.78125	...
......	19	0.296875	....	......	51	0.796875	...
10	20	0.3125	5/16	26	52	0.8125	
......	21	0.328125	....	......	53	0.828125	...
11	22	0.34375	....	27	54	0.84375	...
......	23	0.359375	....	......	55	0.859375	...
12	24	0.375	3/8	28	56	0.875	
......	25	0.390625	....	......	57	0.890625	...
13	26	0.40625	....	29	58	0.90625	...
......	27	0.421875	....	......	59	0.921875	...
14	28	0.4375	7/16	30	60	0.9375	1
......	29	0.453125	....	......	61	0.953125	....
15	30	0.46875	....	31	62	0.96875	....
......	31	0.484375	....	......	63	0.984375	....
16	32	0.5	1/2	32	64	1.	1

Nautical Measures

A nautical or sea-mile is the length of a minute of longitude of the ear he equator at the level of the sea. It is assumed that 6086.07 ft = 1.1 tatute or land-miles by the United States Coast Survey.

3 nautical miles = 1 league

Miscellaneous Measures

1 palm = 3 inches	1 span = 9 inches
1 hand = 4 inches	1 meter = 3.2809 feet

Measures of Surface

144 square inches = 1 square foot
9 square feet = 1 square yard = 1 296 square inches
100 square feet = 1 square (architects' measure)

Land Measure

30¼ square yards = 1 square rod
40 square rods = 1 square rood = 1 210 square yards
4 square roods = 1 acre = 4 840 square yards
10 square chains = 160 square rods
640 acres = 1 square mile = 3 097 600 square yards =
2 560 square roods
208.71 feet square = 1 acre = 43560 square feet

A section of land is a square mile, and a QUARTER-SECTION is 160 acres

Measures of Volume

A gallon, liquid measure = 231 cubic inches, and contains 8.339 avoirdupois pounds of distilled water at 39.8° F., or 58 333 grains
A cubic foot contains 7.48 liquid gallons, or 6.428 dry gallons
A gallon, dry measure = 268.8 cubic inches
A bushel (Winchester) contains 2150.42 cubic inches, or 77.627 pounds distilled water at 39.8° F.
A heaped bushel contains 2747.715 cubic inches

Dry Measure

2 pints = 1 quart = 67.2 cubic inches
4 quarts = 1 gallon = 8 pints = 268.8 cubic inches
2 gallons = 1 peck = 16 pints = 8 quarts = 537.6 cubic inches
4 pecks = 1 bushel = 64 pints = 32 quarts = 8 gallons
= 2150.42 cubic inches
1 cord of wood = 128 cubic feet

Liquid Measure

4 gills = 1 pint = 16 fluid ounces
2 pints = 1 quart = 8 gills = 32 fluid ounces
4 quarts = 1 gallon = 32 gills = 8 pints = 128 fluid ounces

In the United States and Great Britain 1 barrel of wine or brandy = 31½ gallons, and contains 4.211 cubic feet.
A hogshead is 63 gallons, but this term is often applied to casks of various capacities.

Cubic Measure

1728 cubic inches = 1 cubic foot
27 cubic feet = 1 cubic yard

In MEASURING WOOD, a pile of wood cut 4 feet long, piled 4 feet high, and 8 feet on the ground, making 128 cubic feet, is called a CORD.
16 cubic feet make one cord-foot.
A PERCH OF STONE is nominally 16½ feet long, 1 foot high and 1½ feet thick, and contains 24¾ cubic feet.

A perch of stone is, however, often computed differently in different l thus, in most if not all of the States and Territories west of the Mi stone-masons figure rubble by the perch of 16½ cubic feet. In Phil 22 cubic feet are called a perch. In Chicago, stone is measured by th 100 cubic feet.

A TON of shipping is 42 cubic feet in Great Britain and 40 cubic fe United States.

Fluid Measure

60 minims	= 1 fluid drachm
8 fluid drachms	= 1 ounce
16 ounces	= 1 pint
8 pints	= 1 gallon

Miscellaneous Measures

Butt of Sherry	= 108 gallons	Puncheon of Brandy	= 110 to 120
Pipe of Port	= 115 gallons	Puncheon of Rum	= 100 to 110
Butt of Malaga	= 105 gallons	Hogshead of Brandy	= 55 to 60
Puncheon of Scotch Whiskey,	= 110 to 130 gallons	Hogshead of Claret	= 46 gallons

Measures of Weight

The standard AVOIRDUPOIS POUND is the weight of 27.7015 cubic i distilled water weighed in air at 39.83° F., with the barometer at 30 It contains 7 000 grains. One pound avoirdupois = 1.2153 pounds troy

Avoirdupois, or Ordinary Commercial Weight

1 drachm	= 27.343 grains	
16 drachms	= 1 ounce	(oz)
16 ounces	= 1 pound	(lb)
100 pounds	= 1 hundredweight	(cwt)
20 hundredweight	= 1 ton	

In collecting duties upon foreign goods at the United States custom- and also in freighting coal and selling it by wholesale,

28 pounds	= 1 quarter
4 quarters, or 112 pounds	= 1 hundredweight
20 hundredweight	= 1 long ton = 2 240 pounds.
A stone	= 14 pounds
A quintal	= 100 pounds

The following measures are sanctioned by custom or law: 1 bushel = cubic feet or 1¼ cubic feet, nearly.

32 pounds of oats	= 1 bushel
45 pounds of Timothy-seed	= 1 bushel
48 pounds of barley	= 1 bushel
56 pounds of rye	= 1 bushel
56 pounds of Indian corn	= 1 bushel
50 pounds of Indian meal	= 1 bushel
60 pounds of wheat	= 1 bushel
60 pounds of clover-seed	= 1 bushel
60 pounds of potatoes	= 1 bushel

56 pounds of butter	= 1 firkin
100 pounds of meal or flour	= 1 sack
100 pounds of grain or flour	= 1 cental
100 pounds of dry fish	= 1 quintal
100 pounds of nails	= 1 cask
196 pounds of flour	= 1 barrel
200 pounds of beef or pork	= 1 barrel
80 pounds of lime	= 1 bushel

Troy Weight

Used in Weighing Gold or Silver

24 grains	= 1 pennyweight	(pwt)
20 pennyweights	= 1 ounce	(oz)
12 ounces	= 1 pound	(lb)

arat of the jewelers, for precious stones, is, in the United States, 3.2 s, but it varies according to different authorities. In London, 3.17 grains, is, 3.18 grains are divided into 4 jewelers' grains. The international is 3.168 grains or 200 milligrams. In troy, apothecaries' and avoirdupois ts, the grain is the same, 1 pound troy being equal to 0.82286 pound dupois.

Apothecaries' Weight

Used in Compounding Medicines and in Putting Up Medical Prescriptions

20 grains (gr)	= 1 scruple (℈)	8 drachms	= 1 ounce (oz)
3 scruples	= 1 drachm (ʒ)	12 ounces	= 1 pound (lb)

Measures of Value

United States Standard

10 mills	= 1 cent	10 dimes	= 1 dollar
10 cents	= 1 dime	10 dollars	= 1 eagle

e standard of gold and silver is 900 parts of pure metal and 100 of alloy 00 parts of coin.
e fineness expresses the quantity of pure metal in 1 000 parts.
e remedy of the mint is the allowance for deviation from the exact standness and weight of coins.

Weights of Coins

Double eagle	= 516	troy grains
Eagle	= 258	troy grains
Dollar (gold)	= 25.8	troy grains
Dollar (silver)	= 412.5	troy grains
Half-dollar	= 192	troy grains
5-cent piece (nickel)	= 77.16	troy grains
3-cent piece (nickel)	= 30	troy grains
Cent (bronze)	= 48	troy grains

Measures of Time

60 seconds	= 1 minute	365 days	= 1 common year
60 minutes	= 1 hour	366 days	= 1 leap-year
24 hours	= 1 day		

A SOLAR DAY is measured by the rotation of the earth upon its axis, with to the sun.

In ASTRONOMICAL COMPUTATIONS and in NAUTICAL TIME the day comme noon, and in the former it is counted throughout the 24 hours.

In CIVIL COMPUTATIONS the day commences at midnight, and is divid two parts of 12 hours each.

A SOLAR YEAR is the time in which the earth makes one revolution aro sun. Its average time, called the MEAN SOLAR YEAR, is 365 days, 5 48 minutes and 49.7 seconds, or nearly 365¼ days.

A MEAN LUNAR MONTH, or LUNATION of the moon, is 29 days, 12 hours, utes, 2 seconds and 5.24 thirds. It is equal, on the average, to 29.53 day

The Calendar, Old and New Style

The JULIAN Calendar was established by Julius Cæsar, 44 B.C., and by day was inserted in every fourth year. This was the same thing as as that the length of the solar year was 365 days and 6 hours, instead of th given above, thus introducing an accumulative error of 11 minutes and onds every year. This calendar was adopted by the church in 325 A.D., Council of Nice. In the year 1582 the annual error of 11 minutes and 12 s had amounted to 10 days, which, by order of Pope Gregory XIII, was supp in the calendar, and the 5th of October reckoned as the 15th. To preve repetition of this error, it was decided to leave out three of the inserte every 400 years, and to make this omission in the years which are not divisible by 400. Thus, of the years 1700, 1800, 1900 and 2000, all of are leap-years according to the Julian Calendar, only the last is a lea according to the REFORMED or GREGORIAN Calendar. This Reformed Ca was not adopted by England until 1752, when 11 days were omitted fr calendar. The two calendars are now often called the OLD STYLE and th STYLE. The latter style is now adopted in every Christian country Russia.

Circular and Angular Measures

USED FOR MEASURING ANGLES AND ARCS, AND FOR DETERMINING L TUDE AND LONGITUDE

60 seconds (″)	= 1 minute	(′)
60 minutes	= 1 degree	(°)
360 degrees	= 1 circumference	(C)

The SECOND is usually subdivided into tenths and hundredths.

A MINUTE of the circumference of the earth is a geographical mile.

The DEGREES of the earth's circumference on a meridian average 69.16 mon miles.

The Metric System

The METRIC SYSTEM is a system of weights and measures based upon called a METER.

The METER was intended to be one ten-millionth part of the distance fro equator to either pole, measured on the earth's surface at the level of the se

names of derived metric denominations are formed by prefixing to the of the primary unit of measure:

Milli, a thousandth	Hecto, one hundred
Centi, a hundredth	Kilo, a thousand
Deci, a tenth	Myria, ten thousand
Deca, ten	

system, first adopted by France, has been extensively adopted by other ies, and is much used in the sciences and the arts. It was legalized in by Congress to be used in the United States, and is already employed by Coast Survey, and, to some extent, by the Mint and the General Post-

Linear Measures

METER is the primary unit of lengths.

10 millimeters (mm)	= 1 centimeter (cm)	= 0.3937 inch
10 centimeters	= 1 decimeter (dm)	= 3.937 inches
10 decimeters	= 1 METER (m)	= 39.37 inches
10 meters	= 1 decameter	= 393.37 inches
10 decameters	= 1 hectometer	= 328 feet 1 inch
10 hectometers	= 1 KILOMETER (km)	= 0.62137 mile
10 kilometers	= 1 myriameter	= 6.2137 miles

METER is used in ordinary measurements; the CENTIMETER, or MILLI-, in reckoning very small distances; and the KILOMETER, for roads or distances.

CENTIMETER is about 2/5 of an inch; a METER is about 3 feet 3 3/8 inches; a METER is about 200 rods, or 5/8 of a mile. (See page 33.)

Measures of Surface

SQUARE METER is the primary unit of ordinary surfaces.

ARE, a square, each of whose sides is ten METERS, is the unit of land res.

100 square millimeters (mm^2) = 1 square centimeter (cm^2) = 0.155 square inch
100 square centimeters = 1 square decimeter = 15.5 square inches
100 square decimeters = 1 square METER (m^2) = 1 550 square inches, or 1.196 square yards

100 centiares, or square meters	= 1 ARE (a)	= 119.6 square yards
100 ares	= 1 hectare (ha)	= 2.471 acres

SQUARE METER, or one CENTIARE, is about 10 3/4 square feet, or 1 1/5 square s, and a HECTARE is about 2 1/2 acres.

Cubic Measure

CUBIC METER, or STERE, is the primary unit of a volume.

1000 cubic millimeters (mm^3) = 1 cubic centimeter (cm^3) = 0.061 cubic inch
1000 cubic centimeters = 1 cubic decimeter (dm^3) = 61.022 cubic inches
1000 cubic decimeters = 1 cubic METER (m^3) = 35.314 cubic feet

STERE is the name given to the cubic meter in measuring wood and timber. th of a stere is a DECISTERE, and ten steres are a DECASTERE.

CUBIC METER, or STERE, is about 1 1/3 cubic yards, or about 2 1/5 cord feet.

Liquid and Dry Measures

The LITER is the primary unit of measures of capacity, and is a cube, whose edges is a tenth of a meter in length.

The HECTOLITER is the unit in measuring large quantities of grain roots and liquids.

10 milliliters (ml)	= 1 centiliter (cl)	= 0.338 fluid ounce
10 centiliters	= 1 deciliter	= 0.845 liquid gill
10 deciliters	= 1 LITER (l)	= 1.0567 liquid quarts
10 liters	= 1 decaliter	= 2.6417 gallons
10 decaliters	= 1 HECTOLITER (hl)	= 2 bushels, 3.35 pecks
10 hectoliters	= 1 kiloliter	= 28 bushels, 1½ pecks

A CENTILITER is about 1/3 of a fluid ounce; a LITER is about 1 1/16 liquid or 9/10 of a dry quart; a HECTOLITER is about 2 5/6 bushels; and a KILO one cubic meter, or stere.

Weights

The GRAM is the primary unit of weights, and is the weight in a vacu cubic centimeter of distilled water at the temperature of 39.2° F.

10 milligrams (mg)	= 1 centigram (cg)	=	0.1543 troy grain
10 centigrams	= 1 decigram (dg)	=	1.543 troy grains
10 decigrams	= 1 GRAM (g)	=	15.432 troy grains
10 grams	= 1 decagram	=	0.3527 avoirdupois oun
10 decagrams	= 1 hectogram	=	3.5274 avoirdupois oun
10 hectograms	= 1 KILOGRAM (kg)	=	2.2046 avoirdupois pou
10 kilograms	= 1 myriagram	=	22.046 avoirdupois pou
10 myriagrams	= 1 quintal (q)	=	220.46 avoirdupois pou
10 quintals	= 1 TONNEAU (t)	=	2204.6 avoirdupois pou

1 kilogram per kilometer = 0.67195 pound per 1 000 feet
1 pound per thousand feet = 1.4882 kilograms per kilometer
1 kilogram per square millimeter = 1 423 pounds per square inch
1 pound per square inch = 0.000743 kilogram per square millimeter

The GRAM is used in weighing gold, jewels, letters and small quanti things. The KILOGRAM, or, for brevity, KILO, is used by grocers; a TONNEAU, or METRIC TON, is used in finding the weight of very heavy artic

A GRAM is about 15½ grains troy; the KILO about 2 1/5 pounds avoird and the METRIC TON, about 2 205 pounds.

A KILO is the weight of a liter of water at its greatest density; and the TON, of a cubic meter of water.

Metric numbers are written with the decimal point (.) at the right figures denoting the unit; thus the expression, 15 meters 3 centimet written, 15.03 m.

When metric numbers are expressed by figures, the part of the express the left of the decimal point is read as the number of the unit, and the p the right, if any, as a number of the lowest denomination indicated, o decimal part of the unit; thus, 46.525 m is read 46 meters and 525 millim or 46 and 525 thousandths meters.

In writing and reading metric numbers, according as the scale is 10, 1 000, each denomination should be allowed one, two or three orders of fi

Metric Conversion Table

The following metric conversion table has been compiled by C. W. and is most convenient in dealing with metric weights and measures:

Millimeters × 0.03937	= inches
Millimeters ÷ 25.4	= inches
Centimeters × 0.3937	= inches
Centimeters ÷ 2.54	= inches
Meters × 39.37	= inches (Act of Congress)
Meters × 3.281	= feet
Meters × 1.094	= yards
Kilometers × 0.621	= miles
Kilometers ÷ 1.6093	= miles
Kilometers × 3280.7	= feet
Square millimeters × 0.0155	= square inches
Square millimeters ÷ 645.1	= square inches
Square centimeters × 0.155	= square inches
Square centimeters ÷ 6.451	= square inches
Square meters × 10.764	= square feet
Square kilometers × 247.1	= acres
Hectares × 2.471	= acres
Cubic centimeters ÷ 16.383	= cubic inches
Cubic centimeters ÷ 3.69	= fluid drachms (U.S. Pharmacopœia)
Cubic centimeters ÷ 29.57	= fluid ounce. (U.S. Pharmacopœia)
Cubic meters × 35.315	= cubic feet
Cubic meters × 1.308	= cubic yards
Cubic meters × 264.2	= gallons (231 cubic inches)
Liters × 61.022	= cubic inches. (Act of Congress)
Liters × 33.84	= fluid ounces. (U.S. Pharmacopœia)
Liters × 0.2642	= gallons (231 cubic inches)
Liters ÷ 3.78	= gallons (231 cubic inches)
Liters ÷ 28.316	= cubic feet
Hectoliters × 3.531	= cubic feet
Hectoliters × 2.84	= bushels (2 150.42 cubic inches)
Hectoliters × 0.131	= cubic yards
Hectoliters × 26.42	= gallons (231 cubic inches)
Grams × 15.432	= grains. (Act of Congress)
Grams × 981	= dynes
Grams (water) ÷ 29.57	= fluid ounces
Grams ÷ 28.35	= ounces avoirdupois
Grams per cubic centimeter ÷ 27.7	= pounds per cubic inch
Joule × 0.7373	= foot-pounds
Kilograms × 2.2046	= pounds
Kilograms × 35.3	= ounces avoirdupois
Kilograms ÷ 1102.3	= tons (2 000 pounds)
Kilograms per sq cm × 14.223	= pounds per square inch
Kilogrammeters × 7.233	= foot-pounds
Kilograms per meter × 0.672	= pounds per square foot
Kilograms per cubic meter × 0.062	= pounds per cubic foot
Kilograms per cheval-vapeur × 2.235	= pounds per horse-power
Kilowatts × 1.34	= horse-power
Watts ÷ 746	= horse-power
Watts × 0.7373	= foot-pounds per second
Calorie × 3.968	= British thermal units (B.T.U.)
Cheval-vapeur × 0.9863	= horse-power
(Centigrade × 1.8) + 32	= degrees Fahrenheit
Francs × 0.193	= dollars
Gravity, Paris	= 980.94 centimeter per second

Metric Conversion Tables. This and the following table fr
worth's Metrical Tables will be found of great convenience in figu
to be executed in Mexico and other countries using the metric syste

Feet Converted into Meters

Feet	0	1	2	3	
0	....	0.304794	0.609589	0.914383	
10	3.047945	3.35274	3.65753	3.96233	
20	6.095890	6.40068	6.70548	7.01027	
30	9.143835	9.44863	9.75342	10.0582	1
40	12.19178	12.4966	12.8014	13.1062	1
50	15.23972	15.5445	15.8493	16.1541	1
60	18.28767	18.5925	18.8973	19.2020	1
70	21.33561	21.6404	21.9452	22 2500	2
80	24.38356	24.6884	24.9931	25.2979	2
90	27.43150	27.7363	28.0411	28.3459	2

Scripture and Ancient Measures and Weights

Scripture Long Measures

	Inches		Feet	Inc
Digit	= 0.912	Cubit	= 1	9.8
Palm	= 3.648	Fathom	= 7	3.5
Span	= 10.944			

Egyptian Long Measures

Nahud cubit = 1 foot 5.71 inches Royal cubit = 1 foot 8.66 i

Grecian Long Measures

	Feet	Inches		Feet
Digit	=	0.7554	Stadium	= 604
Pous (foot)	= 1	0.0875	Mile	= 4 835
Cubit	= 1	1.5984⅜		

Jewish Long Measures

Cubit	= 1.824 feet	Mile	= 7 296 f
Sabbath-day's journey	= 3 648 feet	Day's journey	= 33.164

Roman Long Measures

	Inches		Feet	In
Digit	= 0.72575	Cubit	= 1	5.
Uncia (inch)	= 0.967	Passus	= 4	10.
Pes (foot)	= 11.604	Mille (millarium)	= 4 842	

Roman Weight

Ancient libbra = 0.7094 pound

Miscellaneous

	Feet		F
Arabian foot	= 1.095	Hebrew foot	= 1.
Babylonian foot	= 1.140	Hebrew cubit	= 1.
Egyptian finger	= 0.06145	Hebrew sacred cubit	= 2.

Metric Conversion Tables

Feet Converted into Meters (Continued)

5	6	7	8	9
1.52397	1.82877	2.13356	2.43836	2.7
4.57192	4.87671	5.18151	5.48630	5.7
7.61986	7.92466	8.22945	8.53425	8.8
10.6678	10.9726	11.2774	11.5822	11.8
13.7158	14.0205	14.3253	14.6301	14.9
16.7637	17.0685	17.3733	17.6781	17.9
19.8116	20.1164	20.4212	20.7260	21.0
22.8596	23.1644	23.4692	23.7740	24.0
25.9075	26.2123	26.5171	26.8219	27.1
28.9555	29.2603	29.5651	29.8699	30.1

...ple. 44 ft = 13.411 meters = 134.11 decimeters = 1 341.1 centimeters = ...
...ers.

...bove-mentioned work contains eighty pages of conversion tables similar...

Inches and Sixteenths Converted into Millimeters

Inches	0	1	2	3	4	
......	....	25.400	50.799	76.199	101.60	12
1/16	1.5875	26.987	52.387	77.786	103.19	12
1/8	3.1749	28.574	53.974	79.374	104.77	13
3/16	4.7624	30.162	55.561	80.961	106.36	13
1/4	6.3499	31.749	57.149	82.549	107.95	13
5/16	7.9374	33.337	58.736	84.136	109.54	13
3/8	9.5248	34.924	60.324	85.723	111.12	13
7/16	11.112	36.512	61.911	87.311	112.71	13
1/2	12.700	38.099	63.499	88.898	114.30	13
9/16	14.287	39.687	65.086	90.486	115.89	14
5/8	15.875	41.274	66.674	92.073	117.47	14
11/16	17.462	42.862	68.261	93.661	119.06	14
3/4	19.050	44.449	69.849	95.248	120.65	14
13/16	20.637	46.037	71.436	96.836	122.24	14
7/8	22.225	47.624	73.024	98.423	123.82	14
15/16	23.812	49.212	74.611	100.01	125.41	15

Inches	6	7	8	9	10	
......	152.40	177.80	203.20	228.60	254.00	27
1/16	153.98	179.38	204.78	230.18	255.58	28
1/8	155.57	180.97	206.37	231.77	257.17	28
3/16	157.16	182.56	207.96	233.36	258.76	28
1/4	158.75	184.15	209.55	234.95	260.35	28
5/16	160.33	185.73	211.13	236.53	261.93	28
3/8	161.92	187.32	212.72	238.12	263.52	28
7/16	163.51	188.91	214.31	239.71	265.11	29
1/2	165.10	190.50	215.90	241.30	266.70	29
9/16	166.68	192.08	217.48	242.88	268.28	29
5/8	168.27	193.67	219.07	244.47	269.87	29
11/16	169.86	195.26	220.66	246.06	271.46	29
3/4	171.45	196.85	222.25	247.65	273.05	29
13/16	173.03	198.43	223.83	249.23	274.63	30
7/8	174.62	200.02	225.42	250.82	276.22	30
15/16	176.21	201.61	227.01	252.41	277.81	30

...meters, move the decimal point THREE figures forward.
...mple. 8 3/16 inches = 207.96 millimeters = 20.796 centimeters = 2.079...
0.20796 meter.

3. GEOMETRY AND MENSURATION

Definitions

A POINT is that which has only position.

A PLANE is a surface in which, any two points being taken, the st joining them will be wholly in the surface.

A CURVED LINE is a line of which no part is straight (Fig. 1).

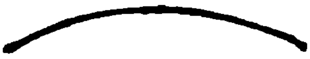


Fig. 1. Curved Line

Fig. 2. Parallel Lines

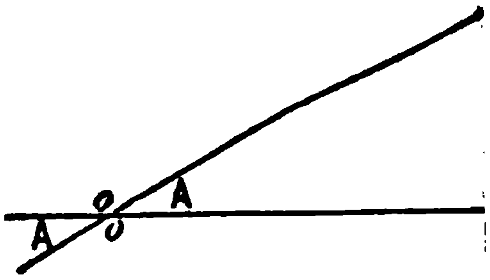


Fig. 3. Angles

PARALLEL LINES are such as are wholly in the same plane, and have direction (Fig. 2).

A BROKEN LINE is a line composed of a series of dashes; thus, —— –

An ANGLE is the opening between two lines meeting at a point, and a RIGHT ANGLE when the two lines are perpendicular to each other, ANGLE when it is less or sharper than a right angle, and an OBTUSE AN it is greater than a right angle. Thus, in Fig. 3,

A A A A are ACUTE ANGLES,

o o o o are OBTUSE ANGLES and *R R R R* are RIGHT ANGLES.

Polygons

A POLYGON is a portion of a plane bounded by straight lines.

A TRIANGLE is a polygon of three sides.

A SCALENE TRIANGLE has none of its sides equal; an ISOSCELES TRIA two of its sides equal; an EQUILATERAL TRIANGLE has all three of its sid

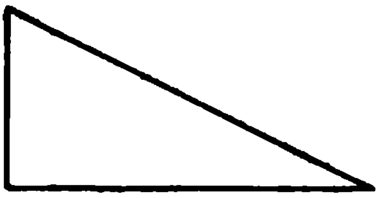


Fig. 4. Right-angled Triangle

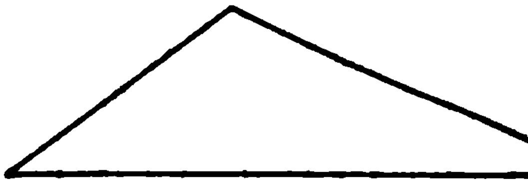


Fig. 5. Scalene Triangle

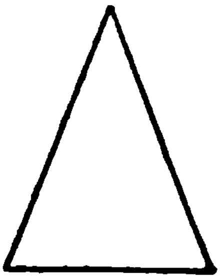


Fig. 6. Isosceles Triangle

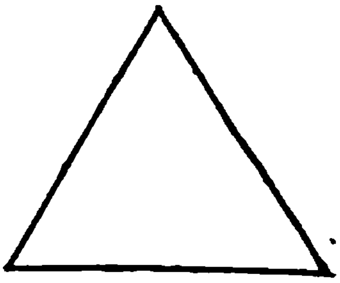
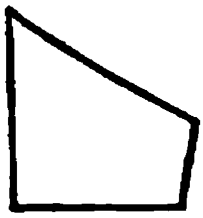


Fig. 7. Equilateral Tria

A RIGHT-ANGLED TRIANGLE is one which has a right angle. The side the right angle is called the HYPOTHENUSE; the side on which the t supposed to stand is called its BASE and the other side, its ALTITUDE.

...ADRILATERAL is a polygon of four sides.

...drilaterals are divided into classes, as follows: the TRAPEZIUM (Fig. 8), ...h has no two of its sides parallel; the TRAPEZOID (Fig. 9), which has two of ...des parallel; and the PARALLELOGRAM (Fig. 10), which is bounded by two ...of parallel sides.



...ig. 8. Trapezium

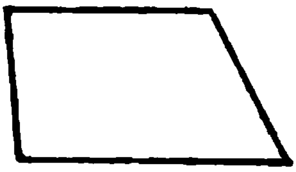


Fig. 9. Trapezoid

Fig. 10. Parallelogram

...parallelogram whose sides are not equal and whose angles are not right ...s is called a RHOMBOID (Fig. 11); when the sides are all equal, but the ...s are not right angles, it is called a RHOMBUS (Fig. 12), and when the angles ...right angles, it is called a RECTANGLE (Fig. 13). A rectangle, all of whose ...s are equal, is called a SQUARE (Fig. 14). Polygons, all of whose sides are ...l, are called REGULAR POLYGONS.

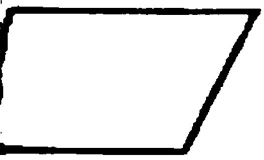


Fig. 11.
Rhomboid

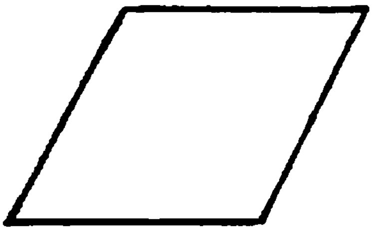


Fig. 12.
Rhombus

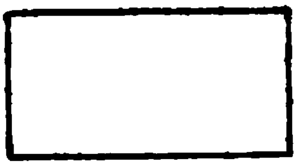


Fig. 13
Rectangle

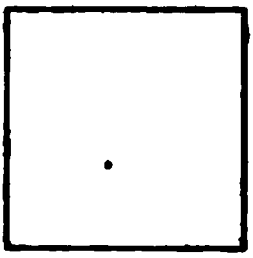


Fig. 14.
Square

...sides the square and equilateral triangles, there are: the PENTAGON (Fig. 15), ...h has five sides; the HEXAGON (Fig. 16), which has six sides; the HEPTAGON ...g. 17), which has seven sides; and the OCTAGON (Fig. 18), which has eight ...s.

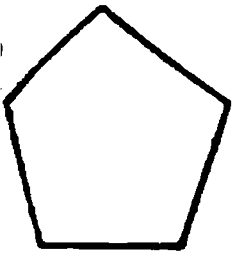


Fig. 15.
Pentagon

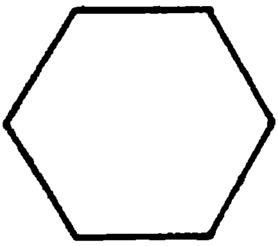


Fig. 16.
Hexagon

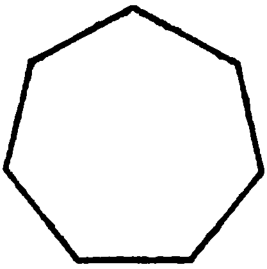


Fig. 17.
Heptagon

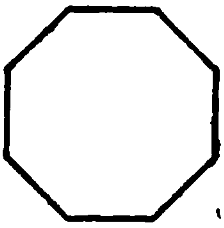


Fig. 18.
Octagon

...he ENNEAGON or NONAGON has nine sides; the DECAGON has ten sides; and ...ODECAGON has twelve sides.

...or all polygons, the side upon which it is supposed to stand is called its ...e; the perpendicular distance from the highest side or angle to the base ...longed, if necessary) is called the ALTITUDE; and a line joining any two ...es not adjacent is called a DIAGONAL.

...PERIMETER is the bounding line of a plane figure.

...CIRCLE is a portion of a plane bounded by a curve, all the points of which ...equidistant from a point within, called the CENTER (Fig. 19).

...he CIRCUMFERENCE is the curve which bounds the circle.

...RADIUS is any straight line drawn from the center to the circumference.

...y straight line drawn through the center to the circumference on each side ...led a DIAMETER.

An ARC of a circle is any part of its circumference.

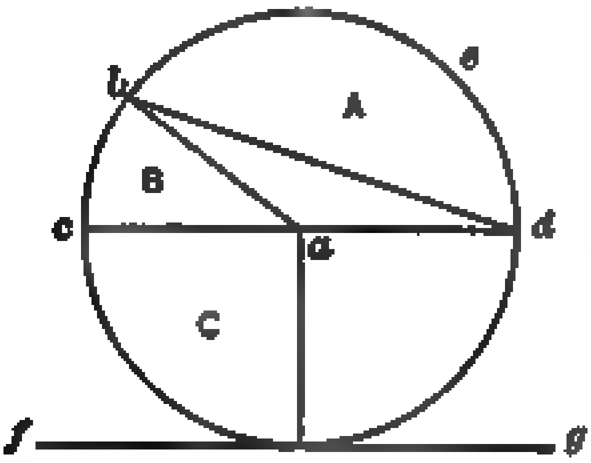


Fig. 19. Circle and Parts

A CHORD is any straight line joi[ning two] points of the circumference, as *bd*, [...]

A SEGMENT is a portion of th[e circle] included between the arc and its c[hord, as] *A*, Fig. 19.

A SECTOR is the space included [between] an arc and two radii drawn to [its ex]tremities, as *B*, Fig. 19. In the fi[gure, *ab*] is a radius, *cd* a diameter and *db* a [chord] SUBTENDING the arc *bed*. A TANG[ENT is a] right line which in passing a curve t[ouches it] without cutting it, as *fg*, Fig. 19.

Volumes

A PRISM is a volume whose ends are equal and parallel polygons and sides are parallelograms.

A prism is TRIANGULAR, RECTANGULAR, etc., according as its ends a[re TRI]ANGLES, RECTANGLES, etc.

A CUBE is a rectangular prism all of whose sides are squares.

A CYLINDER is a volume of uniform diameter, bounded by a curved [surface] and two equal and opposite parallel circles.

A PYRAMID is a volume whose base is a polygon and whose sides are t[riangles] meeting in a point called the VERTEX. A pyramid is triangular, quadra[ngular,] etc., according as its base is a triangle, quadrilateral, etc.

A CONE is a volume whose base is a circle, from which the remaining surface tapers uniformly to a point or vertex (Fig. 20).

A CONIC SECTION is the plane figure made by a plane cutting a cone.

An ELLIPSE is the section of a cone cut by a plane passing obliquely through both sides, as at *ab*, Fig. 21.

A PARABOLA is a section of a cone cut by a plane parallel to its side, as at *cd*.

Fig. 20. Cone

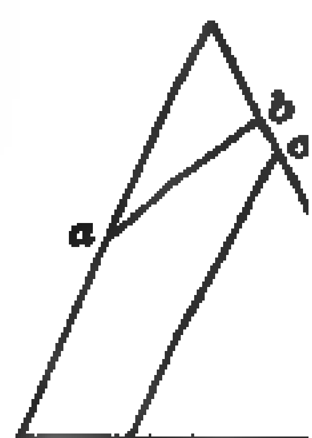


Fig. 21. Cone with Sectio[ns]

A HYPERBOLA is a section of a cone cut by a plane making a greate[r angle] with the base than that made by the side of the cone, as at *eh*.

In the ellipse, the TRANSVERSE AXIS, or LONG DIAMETER, is the longe[st line] that can be drawn in it The CONJUGATE AXIS, or SHORT DIAMETER, is [the line] drawn through the center at right-angles to the long diameter.

A FRUSTUM OF A PYRAMID or CONE is that which remains after cutting [off the] upper part of it by a plane parallel to the base.

A SPHERE is a volume bounded by a curved surface, all points of whi[ch are] equidistant from a point within, called the center.

Mensuration treats of the measurement of lines, surfaces and volum[es].

Rules

To compute the area of a square, a rectangle, a rhombus or a rhomboid.

Rule. Multiply the length by the breadth or height. Thus, in Figs. [...] or 24, the area = $ab \times bc$.

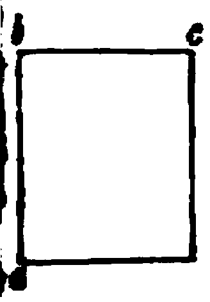


Fig. 22. Square

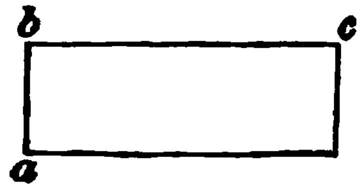


Fig. 23. Rectangle

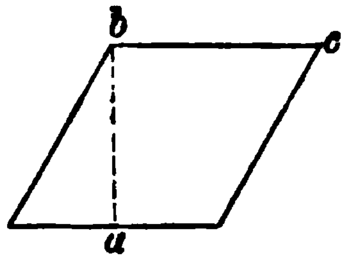


Fig. 24. Parallelogram

compute the area of a triangle.

le. Multiply the base by the altitude and divide by 2. Thus, in Fig. 25, area of $abc = \dfrac{ab \times cd}{2}$.

find the length of the hypothenuse of a right-angled triangle when both are known.

le. Square the length of each of the sides making the right angle, add squares together and take the square root of their sum. Thus (Fig. 26), length of $ac = 3$, and of $bc = 4$; then

$$ab = \sqrt{3^2 + 4^2} = \sqrt{9 + 16} = \sqrt{25}$$

$$\sqrt{25} = 5, \quad \text{or} \quad ab = 5$$

find the length of the base or altitude of a right-angled triangle when length of the hypothenuse and one side is known.

le. From the square of the length of the hypothenuse subtract the square the length of the other side and take the square root of the remainder.

find the area of a trapezium (Fig. 27).

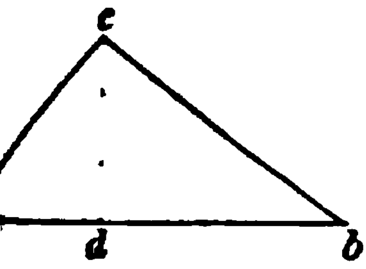


Fig. 25.
Scalene Triangle

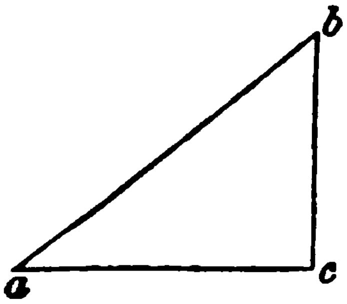


Fig. 26.
Right-angled Triangle

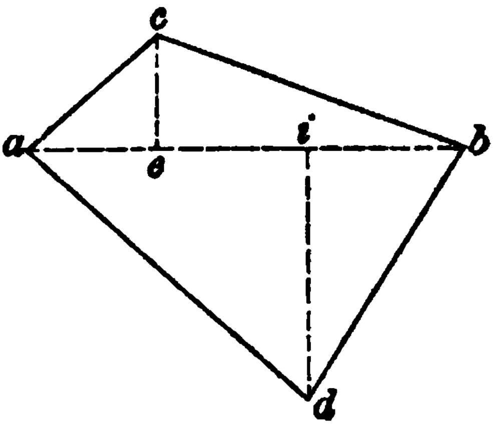


Fig. 27.
Trapezium

le. Multiply the diagonal by the sum of the two perpendiculars falling on it from the opposite angles and divide the product by 2. Thus,

$$\frac{ab \times (ce + di)}{2} = \text{area}$$

find the area of a trapezoid (Fig. 28).

le. Multiply the sum of the two parallel sides by the perpendicular distance between them and divide the product by 2.

compute the area of an irregular polygon.

le. Divide the polygon into triangles by means of diagonal lines and then together the areas of all the triangles, as A, B and C (Fig. 29).

To find the area of a regular polygon.

Rule. Multiply the length of a side by the perpendicular distance
center (as *ao*, Fig. 30), multiply that product by the number of sides and
the result by 2.

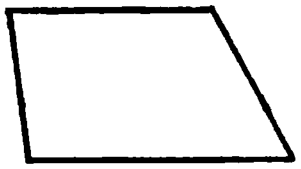


Fig. 28. Trapezoid

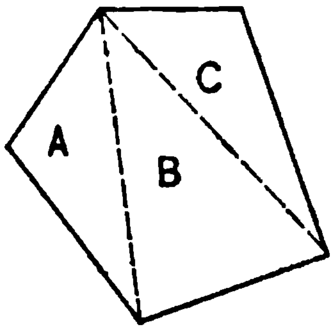


Fig. 29. Irregular Polygon

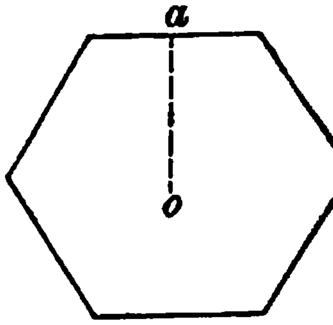


Fig. 30. Regular Poly

**To compute the area of a regular polygon when the length, only, of
is given.**

Rule. Multiply the square of the side by the multiplier opposite the na
the polygon in column *A* of the following table:

Table of Factors for Determining the Elements of Polygons

Name of polygon	Number of sides	*A* Factor for area	*B* Factor for radius of circum-scribing circle	*C* Factor for length of the sides	*L* Facto radiu inscr cir
Triangle...........	3	0.433013	0.5773	1.732	0.2
Tetragon...........	4	1	0.7071	1.4142	0.5
Pentagon...........	5	1.720477	0.8506	1.1756	0.6
Hexagon...........	6	2.598076	1	1	0 8
Heptagon..........	7	3.633912	1.1524	0.8677	1.0
Octagon............	8	4.828427	1.3066	0.7653	1.2
Nonagon...........	9	6.181824	1.4619	0.684	1.3
Decagon...........	10	7.694209	1.618	0.618	1.5
Undecagon.........	11	9.36564	1.7747	0.5634	1.7
Dodecagon.........	12	11.196152	1.9319	0.5176	1.8

**To compute the radius of a circle circumscribed about a regular polygon
the length, only, of a side is given.**

Rule. Multiply the length of a side of the polygon by the number in c
B of table.

Example. What is the radius of a circle that will contain a hexago
length of one side being 5 in?

Solution. $5 \times 1 = 5$ in.

**To compute the length of a side of a regular polygon inscribed in a
circle, when the radius of the circle is given.**

Rule. Multiply the radius of the circle by the number opposite the na
the polygon in column *C* of table.

Example. What is the length of the side of a pentagon contained in a
8 ft in diameter?

Solution. 8 ft diameter $\div 2 = 4$ ft radius; $4 \times 1.1756 = 4.7024$ ft.

To compute the length of a side of a regular polygon, when the radius of the inscribed circle is given.

Rule. Divide the radius of the inscribed circle by the number opposite the name of the polygon in column *D* of table.

To compute the radius of a circle that can be inscribed in a given regular polygon, when the length of a side is given.

Rule. Multiply the length of a side of the polygon by the number opposite the name of the polygon in column *D*.

Example. What is the radius of the circle that can be inscribed in an octagon, the length of one side being 6 in?

Solution. $6 \times 1.2071 = 7.2426$ in.

Circles

To compute the circumference of a circle.

Rule. Multiply the diameter by 3.1416. For many purposes, the multiplier $3\frac{1}{7}$ gives sufficiently accurate results.

Example. What is the circumference of a circle 7 in in diameter?

Solution. $7 \times 3.1416 = 21.9912$ in, or $7 \times 3\frac{1}{7} = 22$ in, the error in this last result being 0.0088 in.

To find the diameter of a circle when the circumference is given.

Rule. Divide the circumference by 3.1416, or for a very close approximate result, multiply by 7 and divide by 22.

To find the radius of an arc when the chord and rise or versed sine are given.

Rule. Square ONE-HALF the CHORD and the RISE; divide the sum of these squares by twice the rise; the result will be the radius.

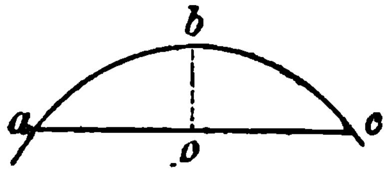


Fig. 31. Circular Arc, Chord and Rise

Example. The length of the chord *ac*, Fig. 31, is 48 in and the rise, *bo*, is 6 in. What is the radius of the arc?

Solution. Radius $= \dfrac{oc^2 + bo^2}{2\,bo} = \dfrac{24^2 + 6^2}{12} = 51$ in

To find the rise or versed sine of a circular arc, when the chord and radius are given.

Rule. Square the radius; also square one-half the chord; subtract the latter from the former and take the square root of the remainder. Subtract the result from the radius and the remainder will be the rise.

Example. A given arc has a radius of 51 in and a chord of 48 in. What is the rise?

Solution. Rise $=$ radius $- \sqrt{\text{radius}^2 - \frac{1}{2}\text{chord}^2} = 51 - \sqrt{2\,601 - 576} = 51 - 45$
$= 6$ in $=$ rise

To compute the area of a circle.

Rule. Multiply the square of the diameter by 0.7854, or multiply the square of the radius by 3.1416.

Example. What is the area of a circle 10 in in diameter?

Solution. $10 \times 10 \times 0.7854 = 78.54$ sq in, or $5 \times 5 \times 3.1416 = 78.54$ sq in.

Tables of Areas and Circumferences of Circles

The following tables will be found very convenient for finding the circumferences and areas of circles.

Areas and Circumferences of Circles

For diameters from 1/10 to 100, advancing by tenths

Dia.	Area	Circum.	Dia.	Area	Circum.	Dia.	Area	Circu
0.0			5.0	19.6350	15.7080	10.0	78.5398	31.41
.1	0.007854	0.31416	.1	20.4282	16.0221	.1	80.1185	31.73
.2	0.031416	0.62832	.2	21.2372	16.3363	.2	81.7128	32.04
.3	0.070686	0.94248	.3	22.0618	16.6504	.3	83.3229	32.35
.4	0.12566	1.2566	.4	22.9022	16.9646	.4	84.9487	32.67
.5	0.19635	1.5708	.5	23.7583	17.2788	.5	86.5901	32.98
.6	0.28274	1.8850	.6	24.6301	17.5929	.6	88.2473	33.30
.7	0.38485	2.1991	.7	25.5176	17.9071	.7	89.9202	33.61
.8	0.50266	2.5133	.8	26.4208	18.2212	.8	91.6088	33.92
.9	0.63617	2.8274	.9	27.3397	18.5354	.9	93.3132	34.24
1.0	0.7854	3.1416	6.0	28.2743	18.8496	11.0	95.0332	34.55
.1	0.9503	3.4558	.1	29.2247	19.1637	.1	96.7689	34.87
.2	1.1310	3.7699	.2	30.1907	19.4779	.2	98.5203	35.18
.3	1.3273	4.0841	.3	31 1725	19.7920	.3	100.2875	35.50
.4	1.5394	4.3982	.4	32.1699	20.1062	.4	102.0703	35.81
.5	1.7671	4.7124	.5	33.1831	20.4204	.5	103.8689	36.12
.6	2.0106	5.0265	.6	34.2119	20.7345	.6	105.6832	36.44
.7	2.2698	5.3407	.7	35.2565	21.0487	.7	107.5132	36.75
.8	2.5447	5.6549	.8	36.3168	21.3628	.8	109.3588	37.07
.9	2.8353	5.9690	.9	37.3928	21.6770	.9	111.2202	37.38
2.0	3.1416	6.2832	7.0	38.4845	21.9911	12.0	113.0973	37.69
.1	3.4636	6.5973	.1	39.5919	22.3053	.1	114.9901	38.01
.2	3.8013	6.9115	.2	40.7150	22.6195	.2	116.8987	38.32
.3	4.1548	7.2257	.3	41.8539	22.9336	.3	118.8229	38.64
.4	4.5239	7.5398	.4	43.0084	23.2478	.4	120.7628	38.95
.5	4.9087	7.8540	.5	44.1786	23.5619	.5	122.7185	39.26
.6	5.3093	8.1681	.6	45.3646	23.8761	.6	124.6898	39.58
.7	5.7256	8.4823	.7	46.5663	24.1903	.7	126.6769	39.89
.8	6.1575	8.7965	.8	47.7836	24.5044	.8	128.6796	40.21
.9	6.6052	9.1106	.9	49.0167	24.8186	.9	130.6981	40.52
3.0	7.0686	9.4248	8.0	50.2655	25.1327	13.0	132.7323	40.84
.1	7.5477	9.7389	.1	51.5300	25.4469	.1	134.7822	41.15
.2	8.0425	10.0531	.2	52.8102	25.7611	.2	136.8478	41.46
.3	8.5530	10.3673	.3	54.1061	26.0752	.3	138.9291	41.78
.4	9.0792	10.6814	.4	55.4177	26.3894	.4	141.0261	42.09
.5	9.6211	10.9956	.5	56.7450	26.7035	.5	143.1388	42.41
.6	10.1788	11.3097	.6	58.0880	27.0177	.6	145.2672	42.72
.7	10.7521	11.6239	.7	59.4468	27.3319	.7	147.4114	43.03
.8	11.3411	11.9381	.8	60.8212	27.6460	.8	149.5712	43.35
.9	11.9459	12.2522	.9	62.2114	27.9602	.9	151.7468	43.66
4.0	12.5664	12.5664	9.0	63.6173	28.2743	14.0	153.9380	43.98
.1	13.2025	12.8805	.1	65.0388	28.5885	.1	156.1450	44.29
.2	13.8544	13.1947	.2	66.4761	28.9027	.2	158.3677	44.61
.3	14.5220	13.5088	.3	67.9291	29.2168	.3	160.6061	44.92
.4	15.2053	13.8230	.4	69.3978	29.5310	.4	162.8602	45.23
.5	15.9043	14.1372	.5	70.8822	29.8451	.5	165.1300	45.55
.6	16.6190	14.4513	.6	72.3823	30.1593	.6	167.4155	45.86
.7	17.3494	14.7655	.7	73.8981	30.4734	.7	169.7167	46.18
.8	18.0956	15.0796	.8	75.4296	30.7876	.8	172.0336	46.49
.9	18.8574	15.3938	.9	76.9769	31.1018	.9	174.3662	46.80

Areas and Circumferences of Circles (Continued)

Advancing by tenths

Dia.	Area	Circum.	Dia.	Area	Circum.	Dia.	Area	Circum.
15.0	176.7146	47.1239	20.0	314.1593	62.8319	25.0	490.8739	78.5398
.1	179.0786	47.4380	.1	317.3087	63.1460	.1	494.8087	78.8540
.2	181.4584	47.7522	.2	320.4739	63.4602	.2	498.7592	79.1681
.3	183.8539	48.0664	.3	323.6547	63.7743	.3	502.7255	79.4823
.4	186.2650	48.3805	.4	326.8513	64.0885	.4	506.7075	79.7965
.5	188.6919	48.6947	.5	330.0636	64.4026	.5	510.7052	80.1106
.6	191.1345	49.0088	.6	333.2916	64.7168	.6	514.7185	80.4248
.7	193.5928	49.3230	.7	336.5353	65.0310	.7	518.7476	80.7389
.8	196.0668	49.6372	.8	339.7947	65.3451	.8	522.7924	81.0531
.9	198.5565	49.9513	.9	343.0698	65.6503	.9	526.8529	81.3672
16.0	301.0619	50.2655	21.0	346.3606	65.9734	26.0	530.9292	81.6814
.1	203.5831	50.5796	.1	349.6671	66.2876	.1	535.0211	81.9956
.2	206.1199	50.8938	.2	352.9894	66.6018	.2	539.1287	82.3097
.3	208.6724	51.2080	.3	356.3273	66.9159	.3	543.2521	82.6239
.4	211.2407	51.5221	.4	359.6809	67.2301	.4	547.3911	82.9380
.5	213.8246	51.8363	.5	363.0503	67.5442	.5	551.5459	83.2522
.6	216.4243	52.1504	.6	366.4354	67.8584	.6	555.7163	83.5664
.7	219.0397	52.4646	.7	369.8361	68.1726	.7	559.9025	83.8805
.8	221.6708	52.7788	.8	373.2526	68.4867	.8	564.1044	84.1947
.9	224.3176	53.0929	.9	376.6848	68.8009	.9	568.3220	84.5088
17.0	226.9801	53.4071	22.0	380.1327	69.1150	27.0	572.5553	84.8230
.1	229.6583	53.7212	.1	383.5963	69.4292	.1	576.8043	85.1372
.2	232.3522	54.0354	.2	387.0756	69.7434	.2	581.0690	85.4513
.3	235.0618	54.3496	.3	390.5707	70.0575	.3	585.3494	85.7655
.4	237.7871	54.6637	.4	394.0814	70.3717	.4	589.6455	86.0796
.5	240.5282	54.9779	.5	397.6078	70.6858	.5	593.9574	86.3938
.6	243.2849	55.2920	.6	401.1500	71.0000	.6	598.2849	86.7080
.7	246.3574	55.6062	.7	404.7078	71.3142	.7	602.6282	87.0221
.8	248.8456	55.9203	.8	408.2814	71.6283	.8	606.9871	87.3363
.9	251.6494	56.2345	.9	411.8707	71.9425	.9	611.3618	87.6504
18.0	254.4690	56.5487	23.0	415.4756	72.2566	28.0	615.7522	87.9646
.1	257.3043	56.8628	.1	419.0963	72.5708	.1	620.1582	88.2788
.2	260.1553	57.1770	.2	422.7327	72.8849	.2	624.5800	88.5929
.3	263.0220	57.4911	.3	426.3848	73.1991	.3	629.0175	88.9071
.4	265.9044	57.8053	.4	430.0526	73.5133	.4	633.4707	89.2212
.5	268.8025	58.1195	.5	433.7361	73.8274	.5	637.9397	89.5354
.6	271.7164	58.4336	.6	437.4354	74.1416	.6	642.4243	89.8495
.7	274.6459	58.7478	.7	441.1503	74.4557	.7	646.9246	90.1637
.8	277.5911	59.0619	.8	444.8809	74.7699	.8	651.4407	90.4779
.9	280.5521	59.3761	.9	448.6273	75.0841	.9	655.9724	90.7920
19.0	283.5267	59.6903	24.0	452.3893	75.3982	29.0	600.5199	91.1062
.1	286.5211	60.0044	.1	456.1671	75.7124	.1	665.0830	91.4203
.2	289.5292	60.3186	.2	459.9606	76.0265	.2	669.6519	91.7345
.3	292.5530	60.6327	.3	463.7698	76.3407	.3	674.2565	92.0487
.4	295.5925	60.9469	.4	467.5947	76.6549	.4	678.8668	92.3628
.5	298.6477	61.2611	.5	471.4352	76.9690	.5	683.4928	92.6770
.6	301.7196	61.5752	.6	475.2916	77.2832	.6	688.1345	92.9911
.7	304.8052	61.8894	.7	479.1636	77.5973	.7	692.7919	93.3053
.8	307.9075	62.2035	.8	483.0513	77.9115	.8	697.4650	93.6195
.9	311.0256	62.5177	.9	486.9547	78.2257	.9	702.1538	93.9336

Areas and Circumferences of Circles (Continued)

Advancing by tenths

Dia.	Area	Circum.	Dia.	Area	Circum.	Dia.	Area	Circum.
30.0	706.8583	94.2478	35.0	962.1128	109.9557	40.0	1256.6371	125.
.1	711.5786	94.5619	.1	967.6184	110.2699	.1	1262.9281	125.
.2	716.3145	94.8761	.2	973.1397	110.5841	.2	1269.2348	126.
.3	721.0662	95.1903	.3	978.6768	110.8982	.3	1275.5573	126.
.4	725.8336	95.5044	.4	984.2296	111.2124	.4	1281.8955	126.
.5	730.6167	95.8186	.5	989.7980	111.5265	.5	1288.2493	127.
.6	735.4154	96.1327	.6	995.3822	111.8407	.6	1294.6189	127.
.7	740.2299	96.4469	.7	1000.9821	112.1549	.7	1301.0042	127.
.8	745.0601	96.7611	.8	1006.5977	112.4690	.8	1307.4052	128.
.9	749.9060	97.0752	.9	1012.2290	112.7832	.9	1313.8219	128.
31.0	754.7676	97.3894	36.0	1017.8760	113.0973	41.0	1320.2543	128.
.1	759.6450	97.7035	.1	1023.5387	113.4115	.1	1326.7024	129.
.2	764.5380	98.0177	.2	1029.2172	113.7257	.2	1333.1663	129.
.3	769.4467	98.3319	.3	1034.9113	114.0398	.3	1339.6458	129.
.4	774.3712	98.6460	.4	1040.6212	114.3540	.4	1346.1410	130.
.5	779.3113	98.9602	.5	1046.3467	114.6681	.5	1352.6520	130.37
.6	784.2672	99.2743	.6	1052.0880	114.9823	.6	1359.1786	130.69
.7	789.2388	99.5885	.7	1057.8449	115.2965	.7	1365.7210	131.00
.8	794.2260	99.9026	.8	1063.6176	115.6106	.8	1372.2791	131.31
.9	799.2290	100.2168	.9	1069.4060	115.9248	.9	1378.8529	131.63
32.0	804.2477	100.5310	37.0	1075.2101	116.2389	42.0	1385.4424	131.94
.1	809.2821	100.8451	.1	1081.0299	116.5531	.1	1392.0476	132.261
.2	814.3322	101.1593	.2	1086.8654	116.8672	.2	1398.6685	152.575
.3	819.3980	101.4734	.3	1092.7166	117.1814	.3	1405.3051	132.889
.4	824.4796	101.7876	.4	1098.5835	117.4956	.4	1411.9574	133.203
.5	829.5768	102.1018	.5	1104.4662	117.8097	.5	1418.6254	133.517
.6	834.6898	102.4159	.6	1110.3645	118.1239	.6	1425.3092	133.831
.7	839.8185	102.7301	.7	1116.2786	118.4380	.7	1432.0086	134.140
.8	844.9628	103.0442	.8	1122.2083	118.7522	.8	1438 7238	134.460
.9	850.1229	103.3584	.9	1128.1538	119.0664	.9	1445.4546	134.774
33.0	855.2986	103.6726	38.0	1134.1149	119.3805	43.0	1452.2012	135.088
.1	860.4902	103.9867	.1	1140.0918	119.6947	.1	1458.9635	135.402
.2	865.6973	104.3009	.2	1146.0844	120.0088	.2	1465 7415	135.716
.3	870.9202	104.6150	.3	1152.0927	120.3230	.3	1472.5352	136.031
.4	876.1588	104.9292	.4	1158.1167	120.6372	.4	1479.3446	136.34
.5	881.4131	105.2434	.5	1164.1564	120.9513	.5	1486 1697	136.659
.6	886.6831	105.5575	.6	1170.2118	121.2655	.6	1493.0105	136.973
.7	891.9688	105.8717	.7	1176.2830	121.5796	.7	1499.8670	137.287
.8	897.2703	106.1858	.8	1182.3698	121.8938	.8	1506.7393	137.601
.9	902.5874	106.5000	.9	1188.4724	122.2080	.9	1513.6272	137.915
34.0	907.9203	106.8142	39.0	1194.5906	122.5221	44.0	1520.5308	138.230
.1	913.2688	107.1283	.1	1200.7246	122.8363	.1	1527.4502	138.544
.2	918.6331	107.4425	.2	1206.8742	123.1504	.2	1534.3853	138.858
.3	924.0131	107.7566	.3	1213.0396	123.4646	.3	1541.3360	139.172
.4	929.4088	108.0708	.4	1219.2207	123.7788	.4	1548.3025	139.486
.5	934.8202	108.3849	.5	1225.4175	124.0929	.5	155 5.247	130.800
.6	940.2473	108.6991	.6	1231.6300	124.4071	.6	1562.2836	140.115
.7	945.6901	109.0133	.7	1237.8582	124.7212	.7	1569.2962	140.429
.8	951.1486	109.3274	.8	1244.1021	125.0354	.8	1576.3255	140.743
.9	956.6228	109.6416	.9	1250.3617	125.3495	.9	1583.3706	141.057

Areas and Circumferences of Circles (Continued)

Advancing by tenths

	Area	Circum.	Dia.	Area	Circum.	Dia.	Area	Circum.
[illegible]	1590.4313	141.3717	50.0	1963.4954	157.0796	55.0	2375.8294	172.7876
[illegible]	1597.5077	141.6858	.1	1971.3572	157.3938	.1	2384.4767	173.1017
[illegible]	1604.5999	142.0000	.2	1979.2348	157.7080	.2	2393.1396	173.4159
[illegible]	1611.7077	142.3142	.3	1987.1280	158.0221	.3	2401.8183	173.7301
[illegible]	1618.8312	142.6283	.4	1995.0370	158.3363	.4	2410.5126	174.0442
[illegible]	1625.9705	142.9425	.5	2002.9617	158.6504	.5	2419.2227	174.3584
[illegible]	1633.1255	143.2566	.6	2010.9020	158.9646	.6	2427.9485	174.6726
[illegible]	1640.2962	143.5708	.7	2018.8581	159.2787	.7	2436.6899	174.9867
[illegible]	1647.4826	143.8849	.8	2026.8299	159.5929	.8	2445.4471	175.3009
.9	1654.6847	144.1991	.9	2034.8174	159.9071	.9	2454.2200	175.6150
[illegible]	1661.9025	144.5133	51.0	2042.8206	160.2212	56.0	2463.0086	175.9292
.1	1669.1360	144.8274	.1	2050.8395	160.5354	.1	2471.8130	176.2433
.2	1676.3853	145.1416	.2	2058.8742	160.8495	.2	2480.6330	176.5575
.3	1683.6502	145.4557	.3	2066.9245	161.1637	.3	2489.4687	176.8717
.4	1690.9308	145.7699	.4	2074.9905	161.4779	.4	2498.3201	177.1858
.5	1698.2272	146.0841	.5	2083.0723	161.7920	.5	2507.1873	177.5000
.6	1705.5392	146.3982	.6	2091.1697	162.1062	.6	2516.0701	177.8141
.7	1712.8670	146.7124	.7	2099.2829	162.4203	.7	2524.9687	178.1283
.8	1720.2105	147.0265	.8	2107.4118	162.7345	.8	2533.8830	178.4425
.9	1727.5697	147.3407	.9	2115.5563	163.0487	.9	2542.8129	178.7566
[illegible]	1734.9445	147.6550	52.0	2123.7166	163.3628	57.0	2551.7586	179.0708
.1	1742.3351	147.9690	.1	2131.8926	163.6770	.1	2560.7200	179.3849
.2	1749.7414	148.2832	.2	2140.0843	163.9911	.2	2569.6971	179.6991
.3	1757.1635	148.5973	.3	2148.2917	164.3053	.3	2578.6899	180.0133
.4	1764.6012	148.9115	.4	2156.5149	164.6195	.4	2587.6985	180.3274
.5	1772.0546	149.2257	.5	2164.7537	164.9336	.5	2596.7227	180.6416
.6	1779.5237	149.5398	.6	2173.0082	165.2479	.6	2605.7626	180.9557
.7	1787.0086	149.8540	.7	2181.2785	165.5619	.7	2614.8183	181.2699
.8	1794.5091	150.1681	.8	2189.5644	165.8761	.8	2623.8896	181.5841
.9	1802.0254	150.4823	.9	2197.8661	166.1903	.9	2632.9767	181.8982
[illegible]	1809.5574	150.7964	53.0	2206.1834	166.5044	58.0	2642.0794	182.2124
.1	1817.1050	151.1106	.1	2214.5165	166.8186	.1	2651.1979	182.5265
.2	1824.6684	151.4248	.2	2222.8653	167.1327	.2	2660.3321	182.8407
.3	1832.2475	151.7389	.3	2231.2298	167.4469	.3	2669.4820	183.1549
.4	1839.8423	152.0531	.4	2239.6100	167.7610	.4	2678.6476	183.4690
.5	1847.4528	152.3672	.5	2248.0059	168.0752	.5	2687.8289	183.7832
.6	1855.0790	152.6814	.6	2256.4175	168.3894	.6	2697.0259	184.0973
.7	1862.7210	152.9956	.7	2264.8448	168.7035	.7	2706.2386	184.4115
.8	1870.3786	153.3097	.8	2273.2879	169.0177	.8	2715.4670	184.7256
.9	1878.0519	153.6239	.9	2281.7466	169.3318	.9	2724.7112	185.0398
[illegible]	1885.7409	153.9380	54.0	2290.2210	169.6460	59.0	2733.9710	185.3540
.1	1893.4457	154.2522	.1	2298.7112	169.9602	.1	2743.2466	185.6681
.2	1901.1662	154.5664	.2	2307.2171	170.2743	.2	2752.5378	185.9823
.3	1908.9024	154.8805	.3	2315.7386	170.5885	.3	2761.8448	186.2964
.4	1916.6543	155.1947	.4	2324.2759	170.9026	.4	2771.1675	186.6106
.5	1924.4218	155.5088	.5	2332.8289	171.2168	.5	2780.5058	186.9248
.6	1932.2051	155.8230	.6	2341.3976	171.5310	.6	2789.8599	187.2389
.7	1940.0042	156.1372	.7	2349.9820	171.8451	.7	2799.2297	187.5531
.8	1947.8189	156.4513	.8	2358.5821	172.1593	.8	2808.6152	187.8672
.9	1955.6493	156.7656	.9	2367.1979	172.4735	.9	2818.0165	188.1814

Areas and Circumferences of Circles

For diameters from 1/10 to 100, advancing by tenths

Dia.	Area	Circum.	Dia.	Area	Circum.	Dia.	Area	Circum.
0.0			5.0	19.6350	15.7080	10.0	78.5398	31.4159
.1	0.007854	0.31416	.1	20.4282	16.0221	.1	80.1185	31.7301
.2	0.031416	0.62832	.2	21.2372	16.3363	.2	81.7128	32.0442
.3	0.070686	0.94248	.3	22.0618	16.6504	.3	83.3229	32.3584
.4	0.12566	1.2566	.4	22.9022	16.9646	.4	84.9487	32.6726
.5	0.19635	1.5708	.5	23.7583	17.2788	.5	86.5901	32.9867
.6	0.28274	1.8850	.6	24.6301	17.5929	.6	88.2473	33.3009
.7	0.38485	2.1991	.7	25.5176	17.9071	.7	89.9202	33.6150
.8	0.50266	2.5133	.8	26.4208	18.2212	.8	91.6088	33.9292
.9	0.63617	2.8274	.9	27.3397	18.5354	.9	93.3132	34.2434
1.0	0.7854	3.1416	6.0	28.2743	18.8496	11.0	95.0332	34.5575
.1	0.9503	3.4558	.1	29.2247	19.1637	.1	96.7689	34.8717
.2	1.1310	3.7699	.2	30.1907	19.4779	.2	98.5203	35.1858
.3	1.3273	4.0841	.3	31 1725	19.7920	.3	100.2875	35.5000
.4	1.5394	4.3982	.4	32.1699	20.1062	.4	102.0703	35.8142
.5	1.7671	4.7124	.5	33.1831	20.4204	.5	103.8689	36.1283
.6	2.0106	5.0265	.6	34.2119	20.7345	.6	105.6832	36.4425
.7	2.2698	5.3407	.7	35.2565	21.0487	.7	107.5132	36.7566
.8	2.5447	5.6549	.8	36.3168	21.3628	.8	109.3588	37.0708
.9	2.8353	5.9690	.9	37.3928	21.6770	.9	111.2202	37.3850
2.0	3.1416	6.2832	7.0	38.4845	21.9911	12.0	113.0973	37.6991
.1	3.4636	6.5973	.1	39.5919	22.3053	.1	114.9901	38.0133
.2	3.8013	6.9115	.2	40.7150	22.6195	.2	116.8987	38.3274
.3	4.1548	7.2257	.3	41.8539	22.9336	.3	118.8229	38.6416
.4	4.5239	7.5398	.4	43.0084	23.2478	.4	120.7628	38.9557
.5	4.9087	7.8540	.5	44.1786	23.5619	.5	122.7185	39.2699
.6	5.3093	8.1681	.6	45.3646	23.8761	.6	124.6898	39.5841
.7	5.7256	8.4823	.7	46.5663	24.1903	.7	126.6769	39.8982
.8	6.1575	8.7965	.8	47.7836	24.5044	.8	128.6796	40.2124
.9	6.6052	9.1106	.9	49.0167	24.8186	.9	130.6981	40.5265
3.0	7.0686	9.4248	8.0	50.2655	25.1327	13.0	132.7323	40.8407
.1	7.5477	9.7389	.1	51.5300	25.4469	.1	134.7822	41.1549
.2	8.0425	10.0531	.2	52.8102	25.7611	.2	136.8478	41.4690
.3	8.5530	10.3673	.3	54.1061	26.0752	.3	138.9291	41.7832
.4	9.0792	10.6814	.4	55.4177	26.3894	.4	141.0261	42.0973
.5	9.6211	10.9956	.5	56.7450	26.7035	.5	143.1388	42.4115
.6	10.1788	11.3097	.6	58.0880	27.0177	.6	145.2672	42.7257
.7	10.7521	11.6239	.7	59.4468	27.3319	.7	147.4114	43.0398
.8	11.3411	11.9381	.8	60.8212	27.6460	.8	149.5712	43.3540
.9	11.9459	12.2522	.9	62.2114	27.9602	.9	151.7468	43.6681
4.0	12.5664	12.5664	9.0	63.6173	28.2743	14.0	153.9380	43.9823
.1	13.2025	12.8805	.1	65.0388	28.5885	.1	156.1450	44.2965
.2	13.8544	13.1947	.2	66.4761	28.9027	.2	158.3677	44.6106
.3	14.5220	13.5088	.3	67.9291	29.2168	.3	160.6061	44.9248
.4	15.2053	13.8230	.4	69.3978	29.5310	.4	162.8602	45.2389
.5	15.9043	14.1372	.5	70.8822	29.8451	.5	165.1300	45.5531
.6	16.6190	14.4513	.6	72.3823	30.1593	.6	167.4155	45.8673
.7	17.3494	14.7655	.7	73.8981	30.4734	.7	169.7167	46.1814
.8	18.0956	15.0796	.8	75.4296	30.7876	.8	172.0336	46.4956
.9	18.8574	15.3938	.9	76.9769	31.1018	.9	174.3662	46.8097

Areas and Circumferences of Circles (Continued)

Advancing by tenths

Area	Circum.	Dia.	Area	Circum.	Dia.	Area	Circum.
4417.8647	235.6194	80.0	5026.5482	251.3274	85.0	5674.5017	267.0354
4429.6533	235.9336	.1	5039.1225	251.6416	.1	5687.8614	267.3495
4441.4580	236.2478	.2	5051.7124	251.9557	.2	5701.2367	267.6637
4453.2753	236.5619	.3	5064.3180	252.2699	.3	5714.6277	267.9779
4465.1142	236.8761	.4	5076.9394	252.5840	.4	5728.0345	268.2920
4476.9659	237.1902	.5	5089.5764	252.8982	.5	5741.4569	268.6062
4488.8332	237.5044	.6	5102.2292	253.2124	.6	5754.8951	268.9203
4500.7163	237.8186	.7	5114.8977	253.5265	.7	5768.3490	269.2345
4512.6151	238.1327	.8	5127.5819	253.8407	.8	5781.8185	269.5486
4524.5296	238.4469	.9	5140.2818	254.1548	.9	5795.3038	269.8628
4536.4598	238.7610	81.0	5152.9973	254.4690	86.0	5808.8048	270.1770
4548.4057	239.0752	.1	5165.7287	254.7832	.1	5822.3215	270.4911
4560.3673	239.3894	.2	5178.4757	255.0973	.2	5835.8539	270.8053
4572.3446	239.7035	.3	5191.2384	255.4115	.3	5849.4020	271.1194
4584.3377	240.0177	.4	5204.0168	255.7256	.4	5862.9659	271.4336
4596.3464	240.3318	.5	5216.8110	256.0398	.5	5876.5454	271.7478
4608.3708	240.6460	.6	5229.6208	256.3540	.6	5890.1407	272.0619
4620.4110	240.9602	.7	5242.4463	256.6681	.7	5903.7516	272.3761
4632.4669	241.2743	.8	5255.2876	256.9823	.8	5917.3783	272.6902
4644.5384	241.5885	.9	5268.1446	257.2966	.9	5931.0206	273.0044
4656.6257	241.9026	82.0	5281.0173	257.6106	87.0	5944.6787	273.3186
4668.7287	242.2168	.1	5293.9056	257.9247	.1	5958.3525	273.6327
4680.8474	242.5310	.2	5306.8097	258.2389	.2	5972.0420	273.9469
4692.9818	242.8451	.3	5319.7295	258.5531	.3	5985.7472	274.2610
4705.1319	243.1592	.4	5332.6650	258.8672	.4	5999.4681	274.5752
4717.2977	243.4734	.5	5345.6162	259.1814	.5	6013.2047	274.8894
4729.4792	243.7876	.6	5358.5832	259.4956	.6	6026.9570	275.2035
4741.6765	244.1017	.7	5371.5658	259.8097	.7	6040.7250	275.5177
4753.8894	244.4159	.8	5384.5641	260.1239	.8	6054.5088	275.8318
4766.1181	244.7301	.9	5397.5782	260.4380	.9	6068.3082	276.1460
4778.3624	245.0442	83.0	5410.6079	260.7522	88.0	6082.1234	276.4602
4790.6225	245.3584	.1	5423.6534	261.0663	.1	6095.9542	276.7743
4802.9983	245.6725	.2	5436.7146	261.3805	.2	6109.8008	277.0885
4815.1897	245.9867	.3	5449.7915	261.6947	.3	6123.6631	277.4026
4827.4969	246.3009	.4	5462.8840	262.0088	.4	6137.5411	277.7168
4839.8198	246.6150	.5	5475.9923	262.3230	.5	6151.4348	278.0309
4852.1584	246.9292	.6	5489.1163	262.6371	.6	6165.3442	278.3451
4864.5128	247.2433	.7	5502.2561	262.9513	.7	6179.2693	278.6593
4876.8828	247.5575	.8	5515.4115	263.2655	.8	6193.2101	278.9740
4889.2685	247.8717	.9	5528.5826	263.5796	.9	6207.1666	279.2876
4901.6699	248.1858	84.0	5541.7694	263.8938	89.0	6221.1389	279.6017
4914.0871	248.5000	.1	5554.9720	264.2079	.1	6235.1268	279.9159
4926.5199	248.8141	.2	5568.1902	264.5221	.2	6249.1304	280.2301
4938.9685	249.1283	.3	5581.4242	264.8363	.3	6263.1498	280.5442
4951.4328	249.4425	.4	5594.6739	265.1514	.4	6277.1840	280.8584
4963.9127	249.7566	.5	5607.9392	265.4646	.5	6291.2356	281.1725
4976.4084	250.0708	.6	5621.2203	265.7787	.6	6305.3021	281.4867
4988.9198	250.3850	.7	5634.5171	266.0929	.7	6319.3813	281.8009
5001.4469	250.6991	.8	5647.8296	266.4071	.8	6333.4822	282.1150
5013.9897	251.0133	.9	5661.1578	266.7212	.9	6347.5958	282.4292

Areas and Circumferences of Circles (Continued)

Advancing by tenths

Dia.	Area	Circum.	Dia.	Area	Circum.	Dia.	Area	Circ
90.0	6361.7251	282.7433	93.5	6866.1471	293.7389	97.0	7389.8113	304.
.1	6375.8701	283.0575	.6	6880.8419	294.0531	.1	7405.0559	305.
.2	6390.0309	283.3717	.7	6895.5524	294.3672	.2	7420.3162	305.
.3	6404.2073	283.6858	.8	6910.2786	294.6814	.3	7435.5922	305.
.4	6418.3995	284.0000	.9	6925.0205	294.9956	.4	7450.8889	305.
.5	6432.6073	284.3141	94.0	6939.7782	295.3097	.5	7466.1913	306.
.6	6446.8309	284.6283	.1	6954.5515	295.6239	.6	7481.5144	306.
.7	6461.0701	284.9425	.2	6969.3106	295.9380	.7	7496.8532	306.
.8	6475.3251	285.2566	.3	6984.1453	296.2522	.8	7512.2078	307.
.9	6489.5958	285.5708	.4	6998.9658	296.5663	.9	7527.5780	307.
91.0	6503.8822	285.8849	.5	7013.8019	296.8805	98.0	7542.9640	307.
.1	6518.1843	286.1991	.6	7028.6538	297.1947	.1	7558.3656	308.
.2	6532.5021	286.5133	.7	7043.5214	297.5088	.2	7573.7830	308.
.3	6546.8356	286.8274	.8	7058.4047	297.8230	.3	7589.2161	308.
.4	6561.1848	287.1416	.9	7073.3033	298.1371	.4	7604.6648	309.
.5	6575.5498	287.4557	95.0	7088.2184	298.4513	.5	7620.1293	309.
.6	6589.9304	287.7699	.1	7103.1488	298.7655	.6	7635.6095	309.
.7	6604.3268	288.0840	.2	7118.1950	299.0796	.7	7651.1054	310.
.8	6618.7388	288.3982	.3	7133.0568	299.3938	.8	7666.6170	310.
.9	6633.1666	288.7124	.4	7148.0343	299.7079	.9	7682.1444	310.
92.0	6647.6101	289.0265	.5	7163.0276	300.0221	99.0	7697.6893	311.
.1	6662.0692	289.3407	.6	7178.0366	300.3363	.1	7713.2461	311.
.2	6676.5441	289.6548	.7	7193.0612	300.6504	.2	7728.8206	311.
.3	6691.0347	289.9690	.8	7208.1016	300.9646	.3	7744.4107	311.
.4	6705.5410	290.2832	.9	7223.1577	301.2787	.4	7760.0166	312.
.5	6720.0630	290.5973	96.0	7238.2295	301.5929	.5	7775.6382	312.
.6	6734.6008	290.9115	.1	7253.3170	301.9071	.6	7791.2754	312.
.7	6749.1542	291.2256	.2	7268.4202	302.2212	.7	7806.9284	313.
.8	6763.7233	291.5398	.3	7283.5391	302.5354	.8	7822.5971	313.
.9	6778.3082	291.8540	.4	7298.6737	302.8405	.9	7838.2815	313.
93.0	6792.9087	292.1681	.5	7313.8240	303.1637	100.0	7853.9816	314.
.1	6807.5250	292.4823	.6	7328.9901	303.4779			
.2	6822.1569	292.7964	.7	7344.1718	303.7920			
.3	6836.8046	293.1106	.8	7359.3693	304.1062			
.4	6851.4680	293.4248	.9	7374.5824	304.4203			

Areas of Circles

Advancing by eighths

Areas

In.	0.0	0.⅛	0.¼	0.⅜	0.½	0.⅝	0.¾	0.⅞
0	0.0	0.0122	0.0490	0.1104	0.1963	0.3068	0.4417	0.6013
1	0.7854	0.9940	1.227	1.484	1.767	2.073	2.405	2.761
2	3.1416	3.546	3.976	4.430	4.908	5.411	5.939	6.491
3	7.069	7.669	8.295	8.946	9.621	10.32	11.04	11.79
4	12.56	13.36	14.18	15.03	15.90	16.80	17.72	18.66
5	19.63	20.62	21.64	22.69	23.75	24.85	25.96	27.10
6	28.27	29.46	30.67	31.91	33.18	34.47	35.78	37.12
7	38.48	39.87	41.28	42.71	44.17	45.66	47.17	48.70
8	50.26	51.84	53.45	55.08	56.74	58.42	60.13	61.86
9	63.61	65.39	67.20	69.02	70.88	72.75	74.66	76.58
10	78.54	80.51	82.51	84.54	86.59	88.66	90.76	92.88
11	95.03	97.20	99.40	101.6	103.8	106.1	108.4	110.7
12	113.0	115.4	117.8	120.2	122.7	125.1	127.6	130.1
13	132.7	135.2	137.8	140.5	143.1	145.8	148.4	151.2
14	153.9	156.6	159.4	162.2	165.1	167.9	170.8	173.7
15	176.7	179.6	182.6	185.6	188.6	191.7	194.8	197.9
16	201.0	204.2	207.3	210.5	213.8	217.0	220.3	223.6
17	226.9	230.3	233.7	237.1	240.5	243.9	247.4	250.9
18	254.4	258.0	261.5	265.1	268.8	272.4	276.1	279.8
19	283.5	287.2	291.0	294.8	298.6	302.4	306.3	310.2
20	314.2	318.1	322.0	326.0	330.0	334.1	338.1	342.2
21	346.4	350.4	354.6	358.8	363.0	367.2	371.5	375.8
22	380.1	384.4	388.8	393.2	397.6	402.0	406.4	410.9
23	415.4	420.0	424.5	429.1	433.7	438.2	443.0	447.6
24	452.3	457.1	461.8	466.6	471.4	476.2	481.1	485.9
25	490.9	495.7	500.7	505.7	510.7	515.7	520.7	525.8
26	530.9	536.0	541.1	546.3	551.5	556.7	562.0	567.2
27	572.5	577.8	583.2	588.5	593.9	599.3	604.8	610.2
28	615.7	621.2	626.7	632.3	637.9	643.5	649.1	654.8
29	660.5	666.2	671.9	677.7	683.4	689.2	695.1	700.9
30	706.9	712.7	718.6	724.6	730.6	736.6	742.6	748.6
31	754.8	760.9	767.0	773.1	779.3	785.5	791.7	798.0
32	804.3	810.6	816.9	823.2	829.6	836.0	842.4	848.8
33	855.3	861.8	868.3	874.9	881.4	888.0	894.6	901.3
34	907.9	914.7	921.3	928.1	934.8	941.6	948.4	955.3
35	962.1	969.0	975.9	982.8	989.8	996.8	1003.8	1010.8
36	1017.9	1025.0	1032.1	1039.2	1046.3	1053.5	1060.7	1068.0
37	1075.2	1082.5	1089.8	1097.1	1104.5	1111.8	1119.2	1126.7
38	1134.1	1141.6	1149.1	1156.6	1164.2	1171.7	1179.3	1186.9
39	1194.6	1202.3	1210.0	1217.7	1225.4	1233.2	1241.0	1248.8
40	1256.6	1264.5	1272.4	1280.3	1288.2	1296.2	1304.2	1312.2
41	1320.3	1328.3	1336.4	1344.5	1352.7	1360.8	1369.0	1377.2
42	1385.4	1393.7	1402.0	1410.3	1418.6	1427.0	1435.4	1443.8
43	1452.2	1460.7	1469.1	1477.6	1486.2	1494.7	1503.3	1511.9
44	1520.5	1529.2	1537.9	1546.6	1555.3	1564.0	1572.8	1581.6
45	1590.4	1599.3	1608.2	1617.0	1626.0	1634.9	1643.9	1652.9

Circumferences of Circles

Advancing by eighths

Circumferences

Dia.	0.0	0.$\frac{1}{8}$	0.$\frac{1}{4}$	0.$\frac{3}{8}$	0.$\frac{1}{2}$	0.$\frac{5}{8}$	0.$\frac{3}{4}$	0
0	0.0	0.3927	0.7854	1.178	1.570	1.963	2.356	2.
1	3.141	3.534	3.927	4.319	4.712	5.105	5.497	5.
2	6.283	6.675	7.069	7.461	7.854	8.246	8.639	9.
3	9.424	9.817	10.21	10.60	10.99	11.38	11.78	12.
4	12.56	12.95	13.35	13.74	14.13	14.52	14.92	15.
5	15.70	16.10	16.49	16.88	17.27	17.67	18.06	18.
6	18.84	19.24	19.63	20.02	20.42	20.81	21.20	21
7	21.99	22.38	22.77	23.16	23.56	23.95	24.34	24
8	25.13	25.52	25.91	26.31	26.70	27.09	27.48	27
9	28.27	28.66	29.05	29.45	29.84	30.23	30.63	31
10	31.41	31.80	32.20	32.59	32.98	33.37	33.77	34
11	34.55	34.95	35.34	35.73	36.12	36.52	36.91	37
12	37.69	38.09	38.48	38.87	39.27	39.66	40.05	40
13	40.84	41.23	41.62	42.01	42.41	42.80	43.19	43
14	43.98	44.37	44.76	45.16	45.55	45.94	46.33	46
15	47.12	47.51	47.90	48.30	48.69	49.08	49.48	49
16	50.26	50.65	51.05	51.44	51.83	52.22	52.62	53.
17	53.40	53.79	54.19	54.58	54.97	55.37	55.76	56.
18	56.54	56.94	57.33	57.72	58.11	58.51	58.90	59.
19	59.69	60.08	60.47	60.86	61.26	61.65	62.04	62.
20	62.83	63.22	63.61	64.01	64.40	64.79	65.18	65.
21	65.97	66.36	66.75	67.15	67.54	67.93	68.32	68.
22	69.11	69.50	69.90	70.29	70.68	71.07	71.47	71.
23	72.25	72.64	73.04	73.43	73.82	74.22	74.61	75.
24	75.39	75.79	76.18	76.57	76.96	77.36	77.75	78.
25	78.54	78.93	79.32	79.71	80.10	80.50	80.89	81.
26	81.68	82.07	82.46	82.85	83.25	83.64	84.03	84.
27	84.82	85.21	85.60	86.00	86.39	86.78	87.17	87.
28	87.96	88.35	88.75	89.14	89.53	89.92	90.32	90.
29	91.10	91.49	91.89	92.28	92.67	93.06	93.46	93.
30	94.24	94.64	95.03	95.42	95.81	96.21	96.60	96.
31	97.39	97.78	98.17	98.57	98.96	99.35	99.75	100.
32	100.53	100.92	101.32	101.71	102.10	102.49	102.89	103.
33	103.67	104.07	104.46	104.85	105.24	105.64	106.03	106.
34	106.81	107.21	107.60	107.99	108.39	108.78	109.17	109.
35	109.96	110.35	110.74	111.13	111.53	111.92	112.31	112.
36	113.10	113.49	113.88	114.28	114.67	115.06	115.45	115.
37	116.24	116.63	117.02	117.42	117.81	118.20	118.60	118.
38	119.38	119.77	120.17	120.56	120.95	121.34	121.74	122.
39	122.52	122.92	123.31	123.70	124.09	124.49	124.88	125.
40	125.66	126.06	126.45	126.84	127.24	127.63	128.02	128.
41	128.81	129.20	129.59	129.98	130.38	130.77	131.16	131.
42	131.95	132.34	132.73	133.13	133.52	133.91	134.30	134.
43	135.09	135.48	135.87	136.27	136.66	137.05	137.45	137.
44	138.23	138.62	139.02	139.41	139.80	140.19	140.59	140.
45	141.37	141.76	142.16	142.55	142.94	143.34	143.73	144.

Areas and Circumferences of Circles

From 1 to 50 Feet

Advancing by one inch

m., in	Area, sq ft	Circum., ft in	Diam., ft in	Area, sq ft	Circum., ft in	Diam., ft in	Area, sq ft	Circum., ft in
0	0.7854	3 1⅝	5 0	19.635	15 8½	9 0	63.6174	28 3¼
1	0.9217	3 4⅞	1	20.2947	15 11⅝	1	64.8006	28 6⅜
2	1.069	3 8	2	20.9656	16 2¾	2	65.9951	28 9½
3	1.2271	3 11	3	21.6475	16 5⅞	3	67.2007	29 ⅝
4	1.3962	4 2⅛	4	22.34	16 9	4	68.4166	29 3¾
5	1.5761	4 5⅜	5	23.0437	17 ⅛	5	69.644	29 7
6	1.7671	4 8½	6	23.7583	17 3¼	6	70.8823	29 10⅛
7	1.9689	4 11⅝	7	24.4835	17 6⅜	7	72.1309	30 1¼
8	2.1816	5 2¾	8	25.2199	17 9⅝	8	73.391	30 4⅜
9	2.4052	5 5⅞	9	25.9672	18 ¾	9	74.662	30 7½
10	2.6398	5 9	10	26.7251	18 3⅞	10	75.9433	30 11⅝
11	2.8852	6 ⅛	11	27.4943	18 7⅛	11	77.2362	31 1⅞
0	3.1416	6 3⅜	6 0	28.2744	18 10⅛	10 0	78.54	31 5
1	3.4087	6 6½	1	29.0649	19 1¼	1	79.854	31 8⅛
2	3.6870	6 9⅝	2	29.8668	19 4⅜	2	81.1795	31 11¼
3	3.976	7 ¾	3	30.6796	19 7½	3	82.516	32 2⅜
4	4.276	7 3⅞	4	31.5029	19 10⅝	4	83.8627	32 5½
5	4.5869	7 7	5	32.3376	20 1⅞	5	85.2211	32 8⅝
6	4.9087	7 10¼	6	33.1831	20 4⅞	6	86.5903	32 11¾
7	5.2413	8 1⅜	7	34.0391	20 8⅛	7	87.9697	33 2⅞
8	5.585	8 4½	8	34.9065	20 11½	8	89.3608	33 6⅛
9	5.9396	8 7⅝	9	35.7847	21 2⅜	9	90.7627	33 9¼
10	6.3049	8 10¾	10	36.6735	21 5½	10	92.1749	34 ⅜
11	6.6813	9 1⅞	11	37.5736	21 8¾	11	93.5986	34 3½
0	7.0686	9 5	7 0	38.4846	21 11⅞	11 0	95.0334	34 6⅝
1	7.4666	9 8¼	1	39.406	22 3	1	96.4783	34 9¾
2	7.8757	9 11⅜	2	40.3388	22 6⅛	2	97.9347	35 ⅞
3	8.2957	10 2½	3	41.2825	22 9¼	3	99.4021	35 4⅛
4	8.7265	10 5⅝	4	42.2367	23 ⅜	4	100.8797	35 7¼
5	9.1683	10 8¾	5	43.2022	23 3½	5	102.3689	35 10⅜
6	9.6211	10 11⅞	6	44.1787	23 6¾	6	103.8691	36 1½
7	10.0846	11 3	7	45.1656	23 9⅞	7	105.3794	36 4½
8	10.5591	11 6⅛	8	46.1638	24 1⅛	8	106.9013	36 7¾
9	11.0446	11 9⅜	9	47.173	24 4⅛	9	108.4342	36 10⅞
10	11.5409	12 ½	10	48.1962	24 7¼	10	109.9772	37 2¾
11	12.0481	12 3⅝	11	49.2236	24 10⅜	11	111.5319	37 5¼
0	12.5664	12 6¾	8 0	50.2656	25 1½	12 0	113.0976	37 8⅜
1	13.0952	12 9⅞	1	51.3178	25 4⅝	1	114.6732	37 11½
2	13.6353	13 1	2	52.3816	25 7⅞	2	116.2607	38 2⅝
3	14.1862	13 4⅛	3	53.4562	25 11	3	117.859	38 5¾
4	14.7479	13 7¼	4	54.5412	26 2⅛	4	119.4674	38 8⅞
5	15.3206	13 10½	5	55.6377	26 5¼	5	121.0876	39 0
6	15.9043	14 1⅝	6	56.7451	26 8⅜	6	122.7187	39 3¼
7	16.4986	14 4⅝	7	57.8628	26 11½	7	124.3598	39 6⅜
8	17.1041	14 7⅞	8	58.992	27 2¾	8	126.0127	39 9½
9	17.7205	14 11	9	60.1321	27 5¾	9	127.6765	40 ⅝
10	18.3476	15 2⅛	10	61.2826	27 9	10	129.3504	40 3¾
11	18.9858	15 5¼	11	62.4445	28 ⅛	11	131.036	40 6⅞

Areas and Circumferences of Circles (Continued)

Area, sq ft	Circum., ft in	Diam., ft in	Area, sq ft	Circum., ft in	Diam., ft in	Area, sq ft	Circ ft
132.7326	40 10	18 0	254.4696	56 6½	23 0	415.4766	72
134.4301	41 1⅛	1	256.8303	56 9⅝	1	418.4915	72
136.1574	41 4⅜	2	259.2033	57 ⅞	2	421.5192	72
137.8867	41 7½	3	261.5872	57 4	3	424.5577	73
139.626	41 10⅝	4	263.9807	57 7⅛	4	427.6055	73
141.3771	42 1⅝	5	266.3864	57 10¼	5	430.6658	73
143.1391	42 4⅞	6	268.8031	58 1⅜	6	433.7371	73
144.9111	42 8	7	271.2293	58 4½	7	436.8175	74
146.6949	42 11⅛	8	273.6678	58 7⅝	8	439.9106	74
148.4896	43 2¼	9	276.1171	58 10¾	9	443.0146	74
150.2943	43 5½	10	278.5761	59 2	10	446.1278	74
152.1109	43 8⅝	11	281.0472	59 5⅛	11	449.2536	75
153.9384	43 11¾	19 0	283.5294	59 8¼	24 0	452.3904	75
155.7758	44 2⅞	1	286.021	59 11½	1	455.5362	75
157.625	44 6	2	288.5249	60 2½	2	458.6948	75
159.4852	44 9⅛	3	291.0307	60 5⅝	3	461.8642	76
161.3553	45 ¼	4	293.5641	60 8¾	4	465.0428	76
163.2373	45 3½	5	296.1107	60 11⅞	5	468.2341	76
165.1303	45 6⅝	6	298.6483	61 3⅛	6	471.4363	76
167.0331	45 9¾	7	301.2054	61 6¼	7	474.6476	77
168.9479	46 ⅞	8	303.7747	61 9½	8	477.8716	77
170.8735	46 4	9	306.355	62 ½	9	481.1065	77
172.8001	46 7⅛	10	308.9448	62 3⅝	10	484.3506	78
174.7565	46 11¼	11	311.5469	62 6¾	11	487.6073	78
176.715	47 1½	20 0	314.16	62 9⅞	25 0	490.875	78
178.6832	47 4⅝	1	316.7824	63 1⅛	1	494.1516	78
180.6634	47 7¾	2	319.4173	63 4¼	2	497.4411	79
182.6545	47 10⅞	3	322.063	63 7⅜	3	500.7415	79
184.6555	48 2½	4	324.7182	63 11½	4	504.051	79
186.6684	48 5⅛	5	327.3858	64 1⅝	5	507.3732	79 1
188.6923	48 8¼	6	330.0643	64 4¾	6	510.7063	80
190.726	48 11⅜	7	332.7522	64 7⅞	7	514.0484	80
192.7716	49 2⅝	8	335.4525	64 11	8	517.4034	80
194.8282	49 5¾	9	338.1637	65 2¼	9	520.7692	80 1
196.8946	49 8⅞	10	340.8844	65 5⅜	10	524.1441	81
198.973	50 0	11	343.6174	65 8¼	11	527.5318	81
201.0624	50 3⅛	21 0	346.3614	65 11⅝	26 0	530.9304	81
203.1615	50 6¼	1	349.1147	66 2¾	1	534.3379	81 1
205.2726	50 9⅝	2	351.8804	66 5⅞	2	537.7583	82
207.3946	51 ½	3	354.6571	66 9	3	541.1896	82
209.5264	51 3¾	4	357.4432	67 ⅛	4	544.6299	82
211.6703	51 6½	5	360.2417	67 3⅜	5	548.083	82 1
213.8251	51 10	6	363.0511	67 6½	6	551.5471	83
215.9896	52 1⅛	7	365.8698	67 9⅝	7	555.0201	83
218.1662	52 4¼	8	368.7011	68 ¾	8	558.5059	83
220.3537	52 7⅜	9	371.5432	68 3⅞	9	562.0027	84
222.551	52 10½	10	374.3947	68 7	10	565.5084	84
224.7608	53 1⅝	11	377.2587	68 10¼	11	569.027	84
226.9806	53 4⅞	22 0	380.1336	69 1⅜	27 0	572.5566	84
229.2105	53 8	1	383.0177	69 4½	1	576.0949	85
231.4525	53 11⅛	2	385.9144	69 7⅝	2	579.6463	85
233.7055	54 2⅛	3	388.822	69 10¾	3	583.2085	85
235.9682	54 5⅜	4	391.7389	70 1⅞	4	586.7796	85 1
238.243	54 8½	5	394.6683	70 5	5	590.3637	86
240.5287	54 11⅝	6	397.6087	70 8¼	6	593.9587	86
242.8241	55 2⅞	7	400.5583	70 11⅛	7	597.5625	86
245.1316	55 6	8	403.5204	71 2½	8	601.1793	86
247.45	55 9⅛	9	406.4935	71 5⅝	9	604.807	87
249.7781	56 ¼	10	409.4759	71 8¾	10	608.4436	87
252.1184	56 3½	11	412.4707	71 11⅞	11	612.0931	87

Areas and Circumferences of Circles (Continued)

Area, sq ft	Circum., ft in	Diam., ft in	Area, sq ft	Circum., ft in	Diam., ft in	Area, sq ft	Circum., ft in
615.7536	87 11⅝	33 0	855.301	103 8	38 0	1134.118	119 4½
619.4238	88 2¾	1	859.624	103 11⅛	1	1139.095	119 7⅝
623.105	88 5¾	2	863.961	104 2¼	2	1144.087	119 10¾
626.7962	88 9	3	868.309	104 5⅜	3	1149.089	120 2
630.5002	89 ⅛	4	872.665	104 8⅝	4	1154.110	120 5⅛
634.2152	89 3¼	5	877.035	104 11¾	5	1159.124	120 8⅜
637.9411	89 6⅜	6	881.415	105 2⅞	6	1164.159	120 11⅜
641.6758	89 9½	7	885.804	105 6	7	1169.202	121 2½
645.4235	90 ⅝	8	890.206	105 9⅛	8	1174.259	121 5⅝
649.1821	90 3¾	9	894.619	106 ¼	9	1179.327	121 8¾
652.9495	90 6⅞	10	899.041	106 3⅜	10	1184.403	121 11⅞
656.73	90 11⅛	11	903.476	106 6⅝	11	1189.493	122 3⅛
660.5214	91 1¼	34 0	907.922	106 9¾	39 0	1194.593	122 6¼
664.3214	91 4⅜	1	912.377	107 ⅞	1	1199.719	122 9½
668.1346	91 7½	2	916.844	107 4	2	1204.824	123 ½
671.9587	91 10⅝	3	921.323	107 7⅛	3	1209.958	123 3⅝
675.7915	92 1¾	4	925.810	107 10¼	4	1215.099	123 6¾
679.6375	92 4⅞	5	930.311	108 1⅜	5	1220.254	123 9⅞
683.4943	92 8⅛	6	934.822	108 4⅝	6	1225.420	124 1⅛
687.3598	92 11¼	7	939.342	108 7¾	7	1230.594	124 4¼
691.2385	93 2⅜	8	943.875	108 10⅞	8	1235.782	124 7⅜
695.1028	93 5½	9	948.419	109 2	9	1240.981	124 10½
699.0263	93 8⅝	10	952.972	109 5⅛	10	1246.188	125 1⅝
702.9377	93 11⅞	11	957.538	109 8¼	11	1251.408	125 4¾
706.86	94 2⅞	35 0	962.115	109 11⅜	40 0	1256.64	125 7⅞
710.791	94 6	1	966.770	110 2⅝	1	1261.879	125 11
714.735	94 9¼	2	971.299	110 5¾	2	1267.133	126 2¼
718.69	95 ⅜	3	975.908	110 8⅞	3	1272.397	126 5⅜
722.654	95 3½	4	980.526	111 0	4	1277.669	126 8½
726.631	95 6⅝	5	985.158	111 3⅛	5	1282.955	126 11⅝
730.618	95 9¾	6	989.803	111 6¼	6	1288.252	127 2¾
734.615	96 ⅞	7	994.451	111 9⅜	7	1293.557	127 5⅞
738.624	96 4	8	999.115	112 ½	8	1298.876	127 9
742.645	96 7¼	9	1003.79	112 3¾	9	1304.206	128 ¼
746.674	96 10⅜	10	1008.473	112 6⅞	10	1309.543	128 3⅜
750.716	97 1½	11	1013.170	112 10	11	1314.895	128 6½
754.769	97 4⅝	36 0	1017.878	113 1⅛	41 0	1320.257	128 9⅝
758.831	97 7¾	1	1022.594	113 4¼	1	1325.628	129 ¾
762.906	97 10⅞	2	1027.324	113 7⅜	2	1331.012	129 3⅞
766.992	98 2	3	1032.064	113 10⅝	3	1336.407	129 7
771.086	98 5⅛	4	1036.813	114 1¾	4	1341.810	129 10⅛
775.191	98 8¼	5	1041.576	114 4⅞	5	1347.227	130 1⅜
779.313	98 11½	6	1046.349	114 8	6	1352.655	130 4½
783.440	99 2⅝	7	1051.130	114 11⅛	7	1358.091	130 7⅝
787.581	99 5¾	8	1055.926	115 2¼	8	1363.541	130 10¾
791.732	99 8⅞	9	1060.731	115 5⅜	9	1369.001	131 1⅞
795.892	100 0	10	1065.546	115 9¼	10	1374.47	131 5
800.065	100 3⅛	11	1070.374	115 11⅝	11	1379.952	131 8⅛
804.25	100 6⅜	37 0	1075.2126	116 2⅞	42 0	1385.446	131 11⅜
808.442	100 9½	1	1080.059	116 6	1	1390.247	132 2½
812.648	101 ⅝	2	1084.920	116 9⅛	2	1396.462	132 5⅝
816.865	101 3¾	3	1089.791	117 ¼	3	1401.988	132 8¾
821.090	101 6⅞	4	1094.671	117 3½	4	1407.522	132 11⅞
825.329	101 10	5	1099.564	117 6½	5	1413.07	133 3
829.579	102 1⅛	6	1104.460	117 9⅝	6	1418.629	133 6⅛
833.837	102 4⅜	7	1109.381	118 ¾	7	1424.195	133 9¼
838.108	102 7½	8	1114.307	118 4	8	1429.776	134 ½
842.391	102 10⅝	9	1119.244	118 7⅛	9	1435.367	134 3⅝
846.681	103 1¾	10	1124.189	118 10¼	10	1440.967	134 6¾
850.985	103 4⅞	11	1129.148	119 1⅜	11	1446.580	134 9⅞

Circumferences of Circles

Advancing by eighths

CIRCUMFERENCES

Dia.	0.0	0.1/8	0.1/4	0.3/8	0.1/2	0.5/8	0.3/4	0.7/8
0	0.0	0.3927	0.7854	1.178	1.570	1.963	2.356	2.748
1	3.141	3.534	3.927	4.319	4.712	5.105	5.497	5.890
2	6.283	6.675	7.068	7.461	7.854	8.246	8.639	9.032
3	9.424	9.817	10.21	10.60	10.99	11.38	11.78	12.17
4	12.56	12.95	13.35	13.74	14.13	14.52	14.92	15.31
5	15.70	16.10	16.49	16.88	17.27	17.67	18.06	18.45
6	18.84	19.24	19.63	20.02	20.42	20.81	21.20	21.5[illegible]
7	21.99	22.38	22.77	23.16	23.56	23.95	24.34	24.7[illegible]
8	25.13	25.52	25.91	26.31	26.70	27.09	27.48	27.8[illegible]
9	28.27	28.66	29.05	29.45	29.84	30.23	30.63	31.0[illegible]
10	31.41	31.80	32.20	32.59	32.98	33.37	33.77	34.1[illegible]
11	34.55	34.95	35.34	35.73	36.12	36.52	36.91	37.3[illegible]
12	37.69	38.09	38.48	38.87	39.27	39.66	40.05	40.4[illegible]
13	40.84	41.23	41.62	42.01	42.41	42.80	43.19	43.5[illegible]
14	43.98	44.37	44.76	45.16	45.55	45.94	46.33	46.7[illegible]
15	47.12	47.51	47.90	48.30	48.69	49.08	49.48	49.8[illegible]
16	50.26	50.65	51.05	51.44	51.83	52.22	52.62	53.0[illegible]
17	53.40	53.79	54.19	54.58	54.97	55.37	55.76	56.1[illegible]
18	56.55	56.94	57.33	57.72	58.11	58.51	58.90	59.2[illegible]
19	59.69	60.08	60.47	60.86	61.26	61.65	62.04	62.4[illegible]
20	62.83	63.22	63.61	64.01	64.40	64.79	65.18	65.5[illegible]
21	65.97	66.36	66.75	67.15	67.54	67.93	68.32	68.7[illegible]
22	69.11	69.50	69.90	70.29	70.68	71.07	71.47	71.8[illegible]
23	72.25	72.64	73.04	73.43	73.82	74.22	74.61	75.0[illegible]
24	75.39	75.79	76.18	76.57	76.96	77.36	77.75	78.1[illegible]
25	78.54	78.93	79.32	79.71	80.10	80.50	80.89	81.2[illegible]
26	81.68	82.07	82.46	82.85	83.25	83.64	84.03	84.4[illegible]
27	84.82	85.21	85.60	86.00	86.39	86.78	87.17	87.5[illegible]
28	87.96	88.35	88.75	89.14	89.53	89.92	90.32	90.7[illegible]
29	91.10	91.49	91.89	92.28	92.67	93.06	93.46	93.8[illegible]
30	94.24	94.64	95.03	95.42	95.81	96.21	96.60	96.9[illegible]
31	97.39	97.78	98.17	98.57	98.96	99.35	99.75	100.1[illegible]
32	100.53	100.92	101.32	101.71	102.10	102.49	102.89	103.2[illegible]
33	103.67	104.07	104.46	104.85	105.24	105.64	106.03	106.4[illegible]
34	106.81	107.21	107.60	107.99	108.39	108.78	109.17	109.5[illegible]
35	109.96	110.35	110.74	111.13	111.53	111.92	112.31	112.7[illegible]
36	113.10	113.49	113.88	114.28	114.67	115.06	115.45	115.8[illegible]
37	116.24	116.63	117.02	117.42	117.81	118.20	118.60	118.9[illegible]
38	119.38	119.77	120.17	120.56	120.95	121.34	121.74	122.1[illegible]
39	122.52	122.92	123.31	123.70	124.09	124.49	124.88	125.2[illegible]
40	125.66	126.06	126.45	126.84	127.24	127.63	128.02	128.4[illegible]
41	128.81	129.20	129.59	129.98	130.38	130.77	131.16	131.[illegible]
42	131.95	132.34	132.73	133.13	133.52	133.91	134.30	134.[illegible]
43	135.09	135.48	135.87	136.27	136.66	137.05	137.45	137.[illegible]
44	138.23	138.62	139.02	139.41	139.80	140.19	140.59	140.[illegible]
45	141.37	141.76	142.16	142.55	142.94	143.34	143.73	144.[illegible]

Table of Circular Arcs

Lengths	Hts	Lengths	Hts	Lengths	Hts	Lengths	Hts
[illegible]	.062	1.01023	.123	[illegible]	.184	1.08797	.245
[illegible]	.063	[illegible]	.124	1.04051	.185	[illegible]	.246
[illegible]	.064	[illegible]	.125	1.04117	.186	[illegible]	.247
[illegible]	.065	1.01123	.126	[illegible]	.187	[illegible]	.248
[illegible]	.066	[illegible]	.127	1.04237	.188	[illegible]	.249
[illegible]	.067	1.01192	.128	[illegible]	.189	[illegible]	.250
[illegible]	.068	[illegible]	.129	[illegible]	.190	[illegible]	.251
[illegible]	.069	1.01264	.130	1.04447	.191	[illegible]	.252
[illegible]	.070	1.01301	.131	1.04515	.192	[illegible]	[illegible]
[illegible]	.071	[illegible]	.132	[illegible]	.193	[illegible]	.254
[illegible]	.072	[illegible]	.133	[illegible]	.194	[illegible]	.255
[illegible]	.073	[illegible]	.134	[illegible]	.195	[illegible]	.256
[illegible]	.074	1.01452	.135	1.04792	.196	[illegible]	.257
[illegible]	.075	1.01492	.136	[illegible]	.197	1.10049	.258
[illegible]	.076	1.01533	.137	[illegible]	.198	1.10147	.259
[illegible]	.077	1.01574	.138	[illegible]	.199	1.10247	.260
[illegible]	.078	1.01614	.139	[illegible]	.200	1.10347	.261
[illegible]	.079	[illegible]	.140	1.05117	.201	1.10447	.262
[illegible]	.080	[illegible]	.141	[illegible]	.202	[illegible]	.263
[illegible]	.081	1.01741	.142	[illegible]	.203	[illegible]	.264
[illegible]	.082	[illegible]	.143	1.05367	.204	[illegible]	.265
[illegible]	.083	[illegible]	.144	1.05441	.205	[illegible]	.266
[illegible]	.084	1.01872	.145	[illegible]	.206	[illegible]	.267
[illegible]	.085	1.01916	.146	1.05591	.207	[illegible]	.268
[illegible]	.086	[illegible]	.147	[illegible]	.208	[illegible]	.269
[illegible]	.087	[illegible]	.148	1.05743	.209	[illegible]	.270
[illegible]	.088	1.02052	.149	[illegible]	.210	1.11274	.271
[illegible]	.089	[illegible]	.150	[illegible]	.211	[illegible]	.272
[illegible]	.090	1.02148	.151	[illegible]	.212	[illegible]	.273
[illegible]	.091	1.02192	.152	1.06051	.213	[illegible]	.274
[illegible]	.092	1.02240	.153	[illegible]	.214	[illegible]	.275
[illegible]	.093	[illegible]	.154	[illegible]	.215	[illegible]	.276
[illegible]	.094	[illegible]	.155	[illegible]	.216	1.12011	.277
[illegible]	.095	[illegible]	.156	[illegible]	.217	1.12115	.278
[illegible]	.096	1.02440	.157	[illegible]	.218	[illegible]	.279
[illegible]	.097	1.02491	.158	[illegible]	.219	[illegible]	.280
[illegible]	.098	[illegible]	.159	1.06611	.220	[illegible]	.281
[illegible]	.099	1.02593	.160	[illegible]	.221	[illegible]	.282
[illegible]	.100	1.02645	.161	[illegible]	.222	[illegible]	.283
[illegible]	.101	[illegible]	.162	[illegible]	.223	[illegible]	.284
[illegible]	.102	[illegible]	.163	[illegible]	.224	[illegible]	.285
[illegible]	.103	[illegible]	.164	[illegible]	.225	[illegible]	.286
[illegible]	.104	[illegible]	.165	1.07100	.226	[illegible]	.287
[illegible]	.105	1.02914	.166	[illegible]	.227	1.13210	.288
[illegible]	.106	1.02970	.167	[illegible]	.228	[illegible]	.289
[illegible]	.107	1.03028	.168	[illegible]	.229	[illegible]	.290
[illegible]	.108	[illegible]	.169	1.07451	.230	[illegible]	.291
[illegible]	.109	1.03139	.170	1.07537	.231	[illegible]	.292
[illegible]	.110	[illegible]	.171	[illegible]	.232	[illegible]	.293
[illegible]	.111	1.03254	.172	1.07711	.233	[illegible]	.294
[illegible]	.112	1.03312	.173	[illegible]	.234	[illegible]	.295
[illegible]	.113	[illegible]	.174	[illegible]	.235	[illegible]	.296
[illegible]	.114	[illegible]	.175	[illegible]	.236	[illegible]	.297
[illegible]	.115	[illegible]	.176	[illegible]	.237	[illegible]	.298
[illegible]	.116	[illegible]	.177	1.08154	.238	[illegible]	.299
[illegible]	.117	[illegible]	.178	[illegible]	.239	[illegible]	.300
[illegible]	.118	[illegible]	.179	1.08337	.240	[illegible]	.301
[illegible]	.119	[illegible]	.180	[illegible]	.241	[illegible]	.302
[illegible]	.120	[illegible]	.181	1.08519	.242	[illegible]	.303
[illegible]	.121	[illegible]	.182	1.08611	.243	1.15070	.304
[illegible]	.122	[illegible]	.183	1.08704	.244	[illegible]	.305

Areas and Circumferences of Circles (Continued)

Diam., ft in	Area, sq ft	Circum., ft in	Diam., ft in	Area, sq ft	Circum., ft in	Diam., ft in	Area, sq ft	Circ ft
13 0	132.7326	40 10	18 0	254.4696	56 6 1/2	23 0	415.4766	72
1	134.4391	41 1 1/8	1	256.8303	56 9 5/8	1	418.4915	72
2	136.1574	41 4 3/8	2	259.2033	57 7/8	2	421.5192	72
3	137.8867	41 7 1/2	3	261.5872	57 4	3	424.5577	73
4	139.626	41 10 5/8	4	263.9807	57 7 1/8	4	427.6055	73
5	141.3771	42 1 5/8	5	266.3864	57 10 1/4	5	430.6658	73
6	143.1391	42 4 7/8	6	268.8031	58 1 3/8	6	433.7371	73
7	144.9111	42 8	7	271.2293	58 4 1/2	7	436.8175	74
8	146.6949	42 11 1/8	8	273.6678	58 7 5/8	8	439.9106	74
9	148.4896	43 2 1/4	9	276.1171	58 10 3/4	9	443.0146	74
10	150.2943	43 5 1/2	10	278.5761	59 2	10	446.1278	74
11	152.1109	43 8 5/8	11	281.0472	59 5 1/8	11	449.2536	75
14 0	153.9384	43 11 3/4	19 0	283.5294	59 8 1/4	24 0	452.3904	75
1	155.7758	44 2 7/8	1	286.021	59 11 1/2	1	455.5362	75
2	157.625	44 6	2	288.5249	60 2 1/2	2	458.6948	75
3	159.4852	44 9 1/8	3	291.0397	60 5 5/8	3	461.8642	76
4	161.3553	45 1/4	4	293.5641	60 8 3/4	4	465.0428	76
5	163.2373	45 3 1/2	5	296.1107	60 11 7/8	5	468.2341	76
6	165.1303	45 6 5/8	6	298.6483	61 3 1/8	6	471.4363	76
7	167.0331	45 9 3/4	7	301.2054	61 6 1/4	7	474.6476	77
8	168.9479	46 7/8	8	303.7747	61 9 1/2	8	477.8716	77
9	170.8735	46 4	9	306.355	62 1/2	9	481.1065	77
10	172.8091	46 7 1/8	10	308.9448	62 3 5/8	10	484.3506	78
11	174.7565	46 11 1/4	11	311.5469	62 6 3/4	11	487.6073	78
15 0	176.715	47 1 1/2	20 0	314.16	62 9 7/8	25 0	490.875	78
1	178.6832	47 4 5/8	1	316.7824	63 1 1/8	1	494.1516	78
2	180.6634	47 7 3/4	2	319.4173	63 4 1/4	2	497.4411	79
3	182.6545	47 10 7/8	3	322.063	63 7 3/8	3	500.7415	79
4	184.6555	48 2 1/2	4	324.7182	63 11 1/2	4	504.051	79
5	186.6684	48 5 1/8	5	327.3858	64 1 5/8	5	507.3732	79
6	188.6923	48 8 1/4	6	330.0643	64 4 3/4	6	510.7063	80
7	190.726	48 11 3/8	7	332.7522	64 7 7/8	7	514.0484	80
8	192.7716	49 2 5/8	8	335.4525	64 11	8	517.4034	80
9	194.8282	49 5 3/4	9	338.1637	65 2 1/4	9	520.7692	80
10	196.8946	49 8 7/8	10	340.8844	65 5 3/8	10	524.1441	81
11	198.973	50 0	11	343.6174	65 8 1/4	11	527.5318	81
16 0	201.0624	50 3 1/8	21 0	346.3614	65 11 5/8	26 0	530.9304	81
1	203.1615	50 6 1/4	1	349.1147	66 2 3/4	1	534.3379	81
2	205.2726	50 9 5/8	2	351.8804	66 5 7/8	2	537.7583	82
3	207.3946	51 1/2	3	354.6571	66 9	3	541.1896	32
4	209.5264	51 3 3/4	4	357.4432	67 1/8	4	544.6299	82
5	211.6703	51 6 1/2	5	360.2417	67 3 3/8	5	548.083	82
6	213.8251	51 10	6	363.0511	67 6 1/2	6	551.5471	83
7	215.9896	52 1 1/8	7	365.8698	67 9 5/8	7	555.0201	83
8	218.1662	52 4 1/4	8	368.7011	68 3/4	8	558.5059	83
9	220.3537	52 7 3/8	9	371.5432	68 3 7/8	9	562.0027	84
10	222.551	52 10 1/2	10	374.3947	68 7	10	565.5084	84
11	224.7608	53 1 5/8	11	377.2587	68 10 1/4	11	569.027	84
17 0	226.9806	53 4 7/8	22 0	380.1336	69 1 3/8	27 0	572.5566	84
1	229.2105	53 8	1	383.0177	69 4 1/2	1	576.0949	85
2	231.4525	53 11 1/8	2	385.9144	69 7 5/8	2	579.6463	85
3	233.7055	54 2 1/8	3	388.822	69 10 3/4	3	583.2085	85
4	235.9682	54 5 3/8	4	391.7389	70 1 7/8	4	586.7796	85
5	238.243	54 8 1/2	5	394.6683	70 5	5	590.3637	86
6	240.5287	54 11 5/8	6	397.6087	70 8 1/4	6	593.9587	86
7	242.8241	55 2 7/8	7	400.5583	70 11 1/8	7	597.5625	86
8	245.1316	55 6	8	403.5204	71 2 1/2	8	601.1793	86
9	247.45	55 9 1/8	9	406.4935	71 5 5/8	9	604.807	87
10	249.7781	56 1/4	10	409.4759	71 8 3/4	10	608.4436	87
11	252.1184	56 3 1/2	11	412.4707	71 11 7/8	11	612.0931	87

Table of Circular Area

		ngths	Hts	Lengths	Hts			
		01021	.123	[illegible]	.184	1.08797	.245	1.15309
		01054	.124	1.04081	.185	1.08890	.246	1.15429
		01088	.125	1.04117	.186	1.08984	.247	1.15549
		01122	.126	1.04151	.187	1.09079	.248	1.15670
		01155	.127	1.04247	.188	1.09174	.249	1.15791
		01193	.128	1.04313	.189	1.09269	.250	1.15912
		01228	.129	1.04380	.190	1.09365	.251	1.16034
		01264	.130	1.04447	.191	1.09461	.252	1.16156
		01300	.131	1.04513	.192	1.09557	.253	1.16279
		01338	.132	1.04580	.193	1.09654	.254	1.16402
		01376	.133	1.04622	.194	1.09752	.255	1.16526
		01414	.134	1.04722	.195	1.09850	.256	1.16650
		01453	.135	1.04792	.196	1.09949	.257	1.16774
		01493	.136	1.04862	.197	1.10048	.258	1.16899
		01533	.137	1.04932	.198	1.10147	.259	1.17024
		01574	.138	1.05003	.199	1.10247	.260	1.17150
		01616	.139	1.05075	.200	1.10347	.261	1.17276
		01656	.140	1.05147	.201	1.10447	.262	1.17403
		01698	.141	1.05220	.202	1.10548	.263	1.17530
		01741	.142	1.05293	.203	1.10650	.264	1.17657
		01784	.143	1.05367	.204	1.10752	.265	1.17784
		01828	.144	1.05411	.205	1.10855	.266	1.17912
		01872	.145	1.05516	.206	1.10958	.267	1.18040
		01916	.146	1.05591	.207	1.11062	.268	1.18169
		01961	.147	1.05667	.208	1.11166	.269	1.18299
		02006	.148	1.05743	.209	1.11279	.270	1.18429
		02052	.149	1.05819	.210	1.11274	.271	1.18559
		02098	.150	1.05897	.211	1.11479	.272	1.18690
		02146	.151	1.05973	.212	1.11584	.273	1.18820
		02192	.152	1.06051	.213	1.11690	.274	1.18951
		02240	.153	1.06130	.214	1.11796	.275	1.19082
		02290	.154	1.06209	.215	1.11904	.276	1.19214
		02339	.155	1.06288	.216	1.12011	.277	1.19346
		02390	.156	1.06369	.217	1.12118	.278	1.19479
		02440	.157	1.06449	.218	1.12225	.279	1.19612
		02491	.158	1.06530	.219	1.12331	.280	1.19746
		02542	.159	1.06611	.220	1.12444	.281	1.19880
		02593	.160	1.06693	.221	1.12554	.282	1.20014
		02646	.161	1.06775	.222	1.12664	.283	1.20149
		02698	.162	1.06858	.223	1.12774	.284	1.20284
		02752	.163	1.06941	.224	1.12885	.285	1.20419
		02806	.164	1.07025	.225	1.12997	.286	1.20555
		02860	.165	1.07100	.226	1.13109	.287	1.20691
		02914	.166	1.07194	.227	1.13219	.288	1.20827
		02970	.167	1.07279	.228	1.13331	.289	1.20964
		03026	.168	1.07365	.229	1.13444	.290	1.21102
		03082	.169	1.07451	.230	1.13557	.291	1.21239
		03139	.170	1.07537	.231	1.13671	.292	1.21377
		03196	.171	1.07624	.232	1.13785	.293	1.21515
		03254	.172	1.07711	.233	1.13900	.294	1.21654
		03312	.173	1.07799	.234	1.14015	.295	1.21794
		03371	.174	1.07888	.235	1.14131	.296	1.21933
		03430	.175	1.07977	.236	1.14247	.297	1.22073
		03490	.176	1.08066	.237	1.14363	.298	1.22213
[illegible]	.116	1.03551	.177	1.08156	.238	1.14480	.299	1.22354
[illegible]	.117	1.03611	.178	1.08246	.239	1.14597	.300	1.22495
[illegible]	.118	1.03672	.179	1.08337	.240	1.14714	.301	1.22636
[illegible]	.119	1.03734	.180	1.08428	.241	1.14832	.302	1.22778
[illegible]	.120	1.03797	.181	1.08519	.242	1.14951	.303	1.22920
[illegible]	.121	1.03860	.182	1.08611	.243	1.15070	.304	1.23063
[illegible]	.122	1.03923	.183	1.08704	.244	1.15189	.305	1.23206

Areas and Circumferences of Circles (Continued)

Diam., ft in	Area, sq ft	Circum., ft in	Diam., ft in	Area, sq ft	Circum., ft in	Diam., ft in	Area, sq ft
43 0	1452.205	135 1	46 0	1661.906	144 6⅛	49 0	1885.745
1	1457.836	135 4⅛	1	1667.931	144 9¼	1	1892.172
2	1463.483	135 7¼	2	1673.97	145 ⅜	2	1898.504
3	1469.14	135 10½	3	1680.02	145 3½	3	1905.037
4	1474.804	136 1⅝	4	1686.077	145 6⅝	4	1911.497
5	1480.483	136 4¾	5	1692.148	145 9⅞	5	1917.961
6	1486.173	136 7⅞	• 6	1698.231	146 1⅛	6	1924.426
7	1491.870	136 11	7	1704.321	146 4⅛	7	1930.919
8	1497.582	137 2⅛	8	1710.425	146 7¼	8	1937.316
9	1503.305	137 5¼	9	1716.541	146 10⅜	9	1943.914
10	1509.035	137 8⅜	10	1722.663	147 1½	10	1050.439
11	1514.779	137 11⅝	11	1728.801	147 4⅝	11	1956.969
44 0	1520.534	138 2¾	47 0	1734.947	147 7¾	50 0	1963.5
1	1526.297	138 5⅞	1	1741.104	147 11	.....	....
2	1532.074	138 9	2	1747.274	148 2⅛	.....	....
3	1537.862	139 ⅛	3	1753.455	148 5¼	.....	....
4	1543.658	139 3¼	4	1759.643	148 8⅜	.....	....
5	1549.478	139. 6⅜	5	1765.845	148 11½	.....	....
6	1555.288	139 9⅝	6	1772.059	149 2⅝	.....	....
7	1561.116	140 ¾	7	1778.28	149 5⅞	.....	....
8	1566.959	140 3⅞	8	1784.515	149 8⅞	.....	....
9	1572.812	140 7½	9	1790.761	150 ⅛	.....	....
10	1578.673	140 10¼	10	1797.015	150 3¼	.....	....
11	1584.549	141 1¼	11	1803.283	150 6⅜	.....	....
45 0	1590.435	141 4⅜	48 0	1809.562	150 9½	.....	....
1	1596.329	141 7½	1	1815.848	151 ⅝	.....	....
2	1002.237	141 10⅝	2	1822.149	151 3¾	.....	....
3	1608.155	142 1⅞	3	1828.460	151 6⅞	.....	....
4	1614.082	142 5	4	1834.779	151 10⅛	.....	....
5	1620.023	142 8⅛	5	1841.173	152 1¼	.....	....
6	1625.974	142 11¼	6	1847.457	152 4⅜	.....	....
7	1631.933	143 2⅜	7	1853.809	152 7½	.....	....
8	1637.907	143 5½	8	1869.175	152 10⅝	.....	....
9	1643.891	143 8¾	0	1866.552	153 1¾	.....	....
10	1649.883	143 11⅞	10	1872.937	153 4⅞	.....	....
11	1655.889	144 3	11	1879.335	153 8⅛	.....	....

Circular Arcs

To find, by the following table, the length of a circular arc when its height, or versed sine is given.

Rule. Divide the height by the chord; find in the column of h number equal to this quotient; take out the corresponding number column of lengths; and multiply this number by the given chord.

Example. The chord of an arc is 80 and its versed sine is 30. W length of the arc?

Solution. $30 \div 80 = 0.375$. The length of an arc for a height of from table, 1.34063. $80 \times 1.34063 = 107.2504 =$ length of arc.

Table of Circular Arcs

Hts	Lengths	Hts	Lengths	Hts	Lengths	Hts	Lengths	Hts	Lengths
[illegible]	1.00001	.062	1.01021	.123	1.03987	.184	1.08797	.245	1.15308
[illegible]	1.00001	.063	1.01054	.124	1.04051	.185	1.08890	.246	1.15428
[illegible]	1.00002	.064	1.01088	.125	1.04116	.186	1.08984	.247	1.15549
[illegible]	1.00004	.065	1.01123	.126	1.04181	.187	1.09079	.248	1.15670
[illegible]	1.00007	.066	1.01158	.127	1.04247	.188	1.09174	.249	1.15791
[illegible]	1.00010	.067	1.01193	.128	1.04313	.189	1.09269	.250	1.15912
[illegible]	1.00013	.068	1.01228	.129	1.04380	.190	1.09365	.251	1.16034
[illegible]	1.00017	.069	1.01264	.130	1.04447	.191	1.09461	.252	1.16156
[illegible]	1.00022	.070	1.01301	.131	1.04515	.192	1.09557	.253	1.16279
[illegible]	1.00027	.071	1.01338	.132	1.04584	.193	1.09654	.254	1.16402
[illegible]	1.00032	.072	1.01376	.133	1.04652	.194	1.09752	.255	1.16526
[illegible]	1.00038	.073	1.01414	.134	1.04722	.195	1.09850	.256	1.16650
[illegible]	1.00045	.074	1.01453	.135	1.04792	.196	1.09949	.257	1.16774
[illegible]	1.00053	.075	1.01493	.136	1.04862	.197	1.10048	.258	1.16899
[illegible]	1.00061	.076	1.01533	.137	1.04932	.198	1.10147	.259	1.17024
[illegible]	1.00069	.077	1.01573	.138	1.05003	.199	1.10247	.260	1.17150
[illegible]	1.00078	.078	1.01614	.139	1.05075	.200	1.10347	.261	1.17276
[illegible]	1.00087	.079	1.01656	.140	1.05147	.201	1.10447	.262	1.17403
[illegible]	1.00097	.080	1.01698	.141	1.05220	.202	1.10548	.263	1.17530
[illegible]	1.00107	.081	1.01741	.142	1.05293	.203	1.10650	.264	1.17657
[illegible]	1.00117	.082	1.01784	.143	1.05367	.204	1.10752	.265	1.17784
[illegible]	1.00128	.083	1.01828	.144	1.05441	.205	1.10855	.266	1.17912
[illegible]	1.00140	.084	1.01872	.145	1.05516	.206	1.10958	.267	1.18040
[illegible]	1.00153	.085	1.01916	.146	1.05591	.207	1.11062	.268	1.18169
[illegible]	1.00167	.086	1.01961	.147	1.05667	.208	1.11165	.269	1.18299
[illegible]	1.00182	.087	1.02006	.148	1.05743	.209	1.11269	.270	1.18429
[illegible]	1.00196	.088	1.02052	.149	1.05819	.210	1.11374	.271	1.18559
[illegible]	1.00210	.089	1.02098	.150	1.05896	.211	1.11479	.272	1.18689
[illegible]	1.00225	.090	1.02145	.151	1.05973	.212	1.11584	.273	1.18820
[illegible]	1.00240	.091	1.02192	.152	1.06051	.213	1.11690	.274	1.18951
[illegible]	1.00256	.092	1.02240	.153	1.06130	.214	1.11796	.275	1.19082
[illegible]	1.00272	.093	1.02289	.154	1.06209	.215	1.11904	.276	1.19214
[illegible]	1.00289	.094	1.02339	.155	1.06288	.216	1.12011	.277	1.19346
[illegible]	1.00307	.095	1.02389	.156	1.06368	.217	1.12118	.278	1.19479
[illegible]	1.00327	.096	1.02440	.157	1.06449	.218	1.12225	.279	1.19612
[illegible]	1.00345	.097	1.02491	.158	1.06530	.219	1.12334	.280	1.19746
[illegible]	1.00364	.098	1.02542	.159	1.06611	.220	1.12444	.281	1.19880
[illegible]	1.00384	.099	1.02593	.160	1.06693	.221	1.12554	.282	1.20014
[illegible]	1.00405	.100	1.02645	.161	1.06775	.222	1.12664	.283	1.20149
[illegible]	1.00426	.101	1.02698	.162	1.06858	.223	1.12774	.284	1.20284
[illegible]	1.00447	.102	1.02752	.163	1.06941	.224	1.12885	.285	1.20419
[illegible]	1.00469	.103	1.02806	.164	1.07025	.225	1.12997	.286	1.20555
[illegible]	1.00492	.104	1.02860	.165	1.07109	.226	1.13108	.287	1.20691
[illegible]	1.00515	.105	1.02914	.166	1.07194	.227	1.13219	.288	1.20827
[illegible]	1.00539	.106	1.02970	.167	1.07279	.228	1.13331	.289	1.20964
[illegible]	1.00563	.107	1.03026	.168	1.07365	.229	1.13444	.290	1.21102
[illegible]	1.00587	.108	1.03082	.169	1.07451	.230	1.13557	.291	1.21239
[illegible]	1.00612	.109	1.03139	.170	1.07537	.231	1.13671	.292	1.21377
[illegible]	1.00638	.110	1.03196	.171	1.07624	.232	1.13785	.293	1.21515
[illegible]	1.00665	.111	1.03254	.172	1.07711	.233	1.13900	.294	1.21654
[illegible]	1.00692	.112	1.03312	.173	1.07799	.234	1.14015	.295	1.21794
[illegible]	1.00720	.113	1.03371	.174	1.07888	.235	1.14131	.296	1.21933
[illegible]	1.00748	.114	1.03430	.175	1.07977	.236	1.14247	.297	1.22073
[illegible]	1.00776	.115	1.03490	.176	1.08066	.237	1.14363	.298	1.22213
[illegible]	1.00805	.116	1.03551	.177	1.08156	.238	1.14480	.299	1.22354
[illegible]	1.00834	.117	1.03611	.178	1.08246	.239	1.14597	.300	1.22495
[illegible]	1.00864	.118	1.03672	.179	1.08337	.240	1.14714	.301	1.22636
[illegible]	1.00895	.119	1.03734	.180	1.08428	.241	1.14832	.302	1.22778
[illegible]	1.00926	.120	1.03797	.181	1.08519	.242	1.14951	.303	1.22920
[illegible]	1.00957	.121	1.03860	.182	1.08611	.243	1.15070	.304	1.23063
[illegible]	1.00989	.122	1.03923	.183	1.08704	.244	1.15189	.305	1.23206

Table of Circular Arcs (Continued)

Hts	Lengths	Hts	Lengths	Hts	Lengths	Hts	Lengths	Hts
.306	1.23349	.345	1.29209	.384	1.35575	.423	1.42402	.462
.307	1.23492	.346	1.29366	.385	1.35744	.424	1.42583	.463
.308	1.23636	.347	1.29523	.386	1.35914	.425	1.42764	.464
.309	1.23781	.348	1.29681	.387	1.36084	.426	1.42945	.465
.310	1.23926	.349	1.29839	.388	1.36254	.427	1.43127	.466
.311	1.24070	.350	1.29997	.389	1.36425	.428	1.43309	.467
.312	1.24216	.351	1.30156	.390	1.36596	.429	1.43491	.468
.313	1.24361	.352	1.30315	.391	1.36767	.430	1.43673	.469
.314	1.24507	.353	1.30474	.392	1.36939	.431	1.43856	.470
.315	1.24654	.354	1.30634	.393	1.37111	.432	1.44039	.471
.316	1.24801	.355	1.30794	.394	1.37283	.433	1.44222	.472
.317	1.24948	.356	1.30954	.395	1.37455	.434	1.44405	.473
.318	1.25095	.357	1.31115	.396	1.37628	.435	1.44589	.474
.319	1.25243	.358	1.31276	.397	1.37801	.436	1.44773	.475
.320	1.25391	.359	1.31437	.398	1.37974	.437	1.44957	.476
.321	1.25540	.360	1.31599	.399	1.38148	.438	1.45142	.477
.322	1.25689	.361	1.31761	.400	1.38322	.439	1.45327	.478
.323	1.25838	.362	1.31923	.401	1.38496	.440	1.45512	.479
.324	1.25988	.363	1.32086	.402	1.38671	.441	1.45697	.480
.325	1.26138	.364	1.32249	.403	1.38846	.442	1.45883	.481
.326	1.26288	.365	1.32413	.404	1.39021	.443	1.46069	.482
.327	1.26437	.366	1.32577	.405	1.39196	.444	1.46255	.483
.328	1.26588	.367	1.32741	.406	1.39372	.445	1.46441	.484
.329	1.26740	.368	1.32905	.407	1.39548	.446	1.46628	.485
.330	1.26892	.369	1.33069	.408	1.39724	.447	1.46815	.486
.331	1.27044	.370	1.33234	.409	1.39900	.448	1.47002	.487
.332	1.27196	.371	1.33399	.410	1.40077	.449	1.47189	.488
.333	1.27349	.372	1.33564	.411	1.40254	.450	1.47377	.489
.334	1.27502	.373	1.33730	.412	1.40432	.451	1.47565	.490
.335	1.27656	.374	1.33896	.413	1.40610	.452	1.47753	.491
.336	1.27810	.375	1.34063	.414	1.40788	.453	1.47942	.492
.337	1.27964	.376	1.34229	.415	1.40966	.454	1.48131	.493
.338	1.28118	.377	1.34396	.416	1.41145	.455	1.48320	.494
.339	1.28273	.378	1.34563	.417	1.41324	.456	1.48509	.495
.340	1.28428	.379	1.34731	.418	1.41503	.457	1.48699	.496
.341	1.28583	.380	1.34899	.419	1.41682	.458	1.48889	.497
.342	1.28739	.381	1.35068	.420	1.41861	.459	1.49079	.498
.343	1.28895	.382	1.35237	.421	1.42041	.460	1.49269	.499
.344	1.29052	.383	1.35406	.422	1.42221	.461	1.49460	.500

Table of Lengths of Circular Arcs whose Radius is 1

Rule. Knowing the measure of the circle and the measure of the arc in minutes and seconds; take from the table the lengths opposite the nu degrees, minutes and seconds in the arc, and multiply their sum by the the circle.

Example. What is the length of an arc subtending an angle of 13 with a radius of 8 ft.

Solution. Length for 13° = 0.2268928
27′ = 0.0078540
8″ = 0.0000388
13° 27′ 8″ = 0.2347856
8
Length of arc = 1.8782848 ft

Lengths of Circular Arcs. Radius = 1

Length	Min	Length.	Deg	Length	Deg	Length
0.0000048	1	0.0002909	1	0.0174533	61	1.06405
0.0000097	2	0.0005818	2	0.0349066	62	1.08210
0.0000145	3	0.0008727	3	0.0523599	63	1.09955
0.0000194	4	0.0011636	4	0.0698132	64	1.11701
0.0000242	5	0.0014544	5	0.0872665	65	1.13446
0.0000291	6	0.0017453	6	0.1047198	66	1.15191
0.0000339	7	0.0020362	7	0.1221730	67	1.16937
0.0000388	8	0.0023271	8	0.1396263	68	1.18682
0.0000436	9	0.0026180	9	0.1570796	69	1.20427
0.0000485	10	0.0029089	10	0.1745329	70	1.22173
0.0000533	11	0.0031998	11	0.1919862	71	1.23918
0.0000582	12	0.0034907	12	0.2094395	72	1.25663
0.0000630	13	0.0037815	13	0.2268928	73	1.27409
0.0000679	14	0.0040724	14	0.2443461	74	1.29154
0.0000727	15	0.0043633	15	0.2617994	75	1.30899
0.0000776	16	0.0046542	16	0.2792527	76	1.32645
0.0000824	17	0.0049451	17	0.2967060	77	1.34390
0.0000873	18	0.0052360	18	0.3141593	78	1.36135
0.0000921	19	0.0055269	19	0.3316126	79	1.37881
0.0000970	20	0.0058178	20	0.3490659	80	1.39626
0.0001018	21	0.0061087	21	0.3665191	81	1.41371
0.0001067	22	0.0063995	22	0.3839724	82	1.43117
0.0001115	23	0.0066904	23	0.4014257	83	1.44862
0.0001164	24	0.0069813	24	0.4188790	84	1.46607
0.0001212	25	0.0072722	25	0.4363323	85	1.48352
0.0001261	26	0.0075631	26	0.4537856	86	1.50098
0.0001309	27	0.0078540	27	0.4712389	87	1.51843
0.0001357	28	0.0081449	28	0.4886922	88	1.53588
0.0001406	29	0.0084358	29	0.5061455	89	1.55334
0.0001454	30	0.0087266	30	0.5235988	90	1.57079
0.0001503	31	0.0090175	31	0.5410521	91	1.58824
0.0001551	32	0.0093084	32	0.5585054	92	1.60570
0.0001600	33	0.0095993	33	0.5759587	93	1.62315
0.0001648	34	0.0098902	34	0.5934119	94	1.64060
0.0001697	35	0.0101811	35	0.6108652	95	1.65806
0.0001745	36	0.0104720	36	0.6283185	96	1.67551
0.0001794	37	0.0107629	37	0.6457718	97	1.69296
0.0001842	38	0.0110538	38	0.6632251	98	1.71042
0.0001891	39	0.0113446	39	0.6806784	99	1.72787
0.0001939	40	0.0116355	40	0.6981317	100	1.74532
0.0001988	41	0.0119264	41	0.7155850	101	1.76278
0.0002036	42	0.0122173	42	0.7330383	102	1.78023
0.0002085	43	0.0125082	43	0.7504916	103	1.79768
0.0002133	44	0.0127991	44	0.7679449	104	1.81514
0.0002182	45	0.0130900	45	0.7853982	105	1.83259
0.0002230	46	0.0133809	46	0.8028515	106	1.85004
0.0002279	47	0.0136717	47	0.8203047	107	1.86750
0.0002327	48	0.0139626	48	0.8377580	108	1.88495
0.0002376	49	0.0142535	49	0.8552113	109	1.90240
0.0002424	50	0.0145444	50	0.8726646	110	1.91986
0.0002473	51	0.0148353	51	0.8901179	111	1.93731
0.0002521	52	0.0151262	52	0.9075712	112	1.95476
0.0002570	53	0.0154171	53	0.9250245	113	1.97222
0.0002618	54	0.0157080	54	0.9424778	114	1.98967
0.0002666	55	0.0159989	55	0.9599311	115	2.00712
0.0002715	56	0.0162897	56	0.9773844	116	2.02458
0.0002763	57	0.0165806	57	0.9948377	117	2.04203
0.0002812	58	0.0168715	58	1.0122910	118	2.05948
0.0002860	59	0.0171624	59	1.0297443	119	2.07694
0.0002909	60	0.0174533	60	1.0471976	120	2.09439

To compute the chord of an arc when the chord of half the arc and the sine are given.

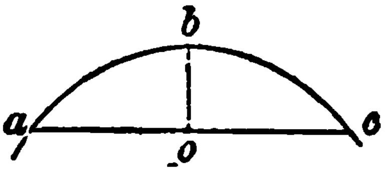


Fig. 32. Circular Arc, Chord and Rise

(The versed sine is the perpendicular *bo*, Fig.

Rule. From the square of the chord of half t subtract the square of the versed sine, and take the square root of the remainder.

Example. The chord of half the arc is 60, a versed sine 36. What is the length of the chord arc?

Solution. $60^2 - 36^2 = 2\,304$; $\sqrt{2\,304} = 48$; and $48 \times 2 = 96$, the chor

To compute the chord of an arc when the diameter and versed sine are gi

Multiply the versed sine by 2 and subtract the product from the dia then subtract the square of the remainder from the square of the diamet take the square root of that remainder.

Example. The diameter of a circle is 100 and the versed sine of an What is the chord of the arc?

Solution. $36 \times 2 = 72$; $100 - 72 = 28$; $100^2 - 28^2 = 9\,216$; $\sqrt{9\,21}$ the chord of the arc.

To compute the chord of half an arc when the chord of the arc and the sine are given.

Rule. Take the square root of the sum of the squares of the versed si of half the chord of the arc.

Example. The chord of an arc is 96 and the versed sine 36. What is the of half the arc?

Solution. $\sqrt{36^2 + 48^2} = 60$.

To compute the chord of half an arc when the diameter and versed si given.

Rule. Multiply the diameter by the versed sine and take the squa of their product.

To compute a diameter.

Rule 1. Divide the square of the chord of half the arc by the versed si

Rule 2. Add the square of half the chord of the arc to the square versed sine and divide this sum by the versed sine.

Example. What is the radius of an arc whose chord is 96 and whose sine is 36?

Solution. $48^2 + 36^2 = 3\,600$; $3\,600 \div 36 = 100$, the diameter; and the $= 50$.

To compute the versed sine.

Rule. Divide the square of the chord of half the arc by the diameter.

To compute the versed sine when the chord of the arc and the diamet given.

Rule. From the square of the diameter subtract the square of the and extract the square root of the remainder; subtract this root from the eter and halve the remainder.

To compute the length of an arc of a circle when the number of degree the radius are given.

Rule 1. Multiply the number of degrees in the arc by 3.1416 multipli the radius and divide by 180. The result will be the length of the arc same unit as the radius.

2. Multiply the radius of the circle by 0.01745 and the product by the in the arc.

ple. The number of degrees in an arc is 60 and the radius is 10 in. the length of the arc in inches?

tion. $10 \times 3.1416 \times 60 = 1884.96$; and $1884.96 \div 180 = 10.47$ in. Or, $0.01745 \times 60 = 10.47$ in.

compute the length of the arc of a circle when the length is given in de-, minutes and seconds.

le. (1) Multiply the number of degrees by 0.01745329 and the product radius. (2) Multiply the number of minutes by 0.00029 and that prod- by the radius. (3) Multiply the number of seconds by 0.0000048 times radius. (4) Add together these three results for the length of the arc. also, table, page 57.)

mple. What is the length of an arc of 60° 10′ 5″, the radius being 4 ft?

tion.
(1) $60° \times 0.01745329 \times 4 = 4.188789$ ft
(2) $10' \times 0.00029 \times 4 = 0.0116$ ft
(3) $5'' \times 0.0000048 \times 4 = 0.000096$ ft
(4) The length of the arc $= 4.200485$ ft

compute the area of a sector of a circle when the degrees of the arc and radius are given (Fig. 33).

The degrees of the arc are the same as the angle *aob*.)

le. Multiply the number of degrees in the arc by area of the whole circle and divide by 360.

mple. What is the area of a sector of a circle whose is 5 and length of arc 60°?

tion. Area of circle $= 10 \times 10 \times 0.7854 = 78.54$

, area of sector $= \dfrac{78.5 \times 60}{360} = 13.09$

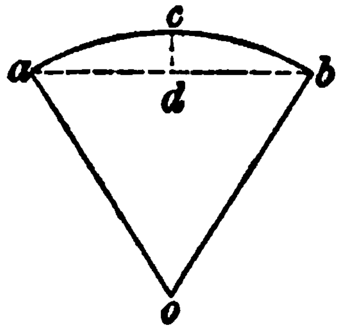


Fig. 33. Sector of Circle

te. If the length of the arc is given in degrees and tes, reduce it to minutes, multiply by the area of the whole circle and divide 600.

compute the area of a sector of a circle when the length of the arc and s are given.

le. Multiply the length of the arc by half the length of the radius. The act is the area.

compute the area of a segment of a circle when the chord and versed sine arc and the radius or diameter of the circle are given.

he versed sine is the distance *cd*, Fig. 33.)

le 1. **When the segment is less than a semicircle.** (1) Find the area of ector having the same arc as the segment. (2) Find the area of a triangle d by the chord of the segment and the radii of the sector. (3) Take the ence of these areas.

le 2. **When the segment is greater than a semicircle.** Find, by the pre- g rule, the area of the lesser portion of the circle and subtract it from the f the whole circle. The remainder will be the area.

compute the area of the surface of a sphere.

e. Multiply the diameter by the circumference. The product will be ea of the surface.

Example. What is the area of the surface of a sphere 10 in in diam

Solution. Circumference of sphere = $10 \times 3.1416 = 31.416$ in; 10
= 314.16 sq in, the area of surface of sphere.

To compute the total area of the surface of a segment of a sphere.

Rule. Multiply the height (*bc*, Fig. 34) by the circumference of th and add the product to the area of the base.

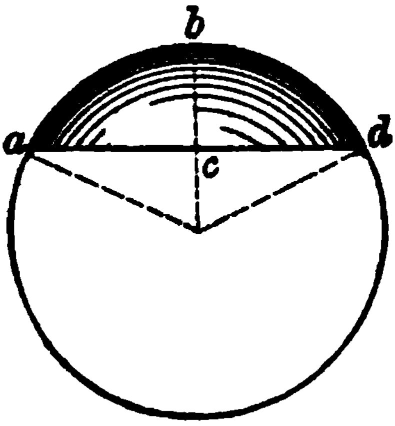


Fig. 34.
Segment of Sphere

To find the area of the base, having the diameter of the sphere and the length of the versed sine of the arc *abd*, find the length of the chord *ad* by the rule on page 58. Having, then, the length of the chord *ad* for the diameter of the base, find the area of the base.

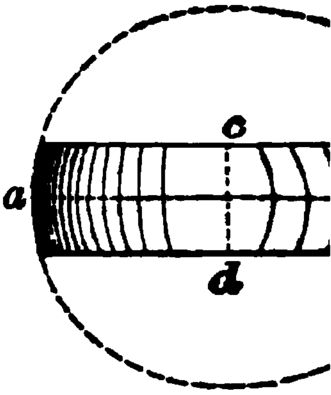


Fig. 35.
Zone of Sphe

Example. The height, *bc*, of a segment *abd*, is 36 in, and the diamete sphere is 100 in (Fig. 34). What is the area of the convex surface and t of the whole surface?

Solution. $100 \times 3.1416 = 314.16$ in, the circumference of sphere
$36 \times 314.16 = 11309.76$ sq in, the area of the convex surfac
$100 - (36 \times 2) = 28$
$\sqrt{100^2 - 28^2} = 96$, the chord *ad*
$96^2 \times 0.7854 = 7238.2464$ sq in, the area of the base
$11309.76 + 7238.2464 = 18548.0064$ sq in, the total area

To compute the total area of the surface of a spherical zone.

Rule. Multiply the height, *cd* (Fig. 35), by the circumference of the for the convex surface and add to it the area of the two ends for the total a

Spheroids, or Ellipsoids of Revolution

Definition. Spheroids, or ellipsoids, are figures generated by the rev of a semiellipse about one of its diameters.

When the revolution is about the long diameter, they are PROLATE when it is about the short diameter, they are OBLATE.

A PROLATE SPHEROID is approximately cigar-shaped and an OBLATE SP is, in form, somewhat like a watch.

To compute the area of the surface of a spheroid.

Let $a = \frac{1}{2}$ the long axis; let $b = \frac{1}{2}$ the short axis;

let
$$\frac{a^2 - b^2}{a^2} = e^2 \quad \text{or} \quad e = \sqrt{\frac{a^2 - b^2}{a^2}}$$

Then, the area of the SURFACE OF THE OBLATE SPHEROID

$$= 2\pi a^2 + \frac{\pi b^2}{e} \log\left(\frac{1 + e}{1 - e}\right)$$

and the area of the SURFACE OF THE PROLATE SPHEROID

$$= 2\pi b^2 + 2\pi ab \frac{\sin^{-1} e}{e}$$

the first formula, NATURAL LOGARITHMS must be used. The natural loga- may be obtained by multiplying the common logarithm by 2.302. The of the expression $\sin^{-1} e$ may be determined by finding the angle whose sine is equal to e and dividing this angle by

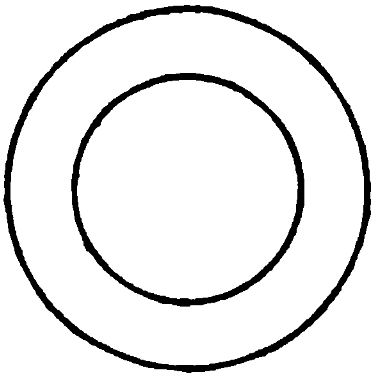


Fig. 36. Circular Ring

Although the above formulas are compli- no simpler rules that give correct results can be

compute the area of the surface of a cylinder.

Multiply the length of the cylinder by the ference of one of the ends and add to the the areas of the two ends.

compute the area of a circular ring (Fig. 36).

Find the area of both circles and subtract the area of the smaller from of the larger; the remainder will be the area of the ring.

compute the area of the surface of a cone.

Multiply the circumference of the base by one-half the slant-height of the cone, for the convex area. Add to this the area of the base, for whole area.

Example. The diameter of the base of a cone is 3 in and the slant-height 15 in. is the area of the surface of the cone?

Solution.	3×3.1416	= 9.4248	= circumference of base
	$9.4248 \times 7\frac{1}{2}$	= 70.686 sq in	= area of convex surface
	$3 \times 3 \times 0.7854$	= 7.068 sq in	= area of base
Area of entire surface of cone		= 77.754 sq in	

compute the area of the surface of the frustum of a cone (Fig. 37).

37. Frustum of Cone

Rule. Multiply the sum of the circumferences of the two ends by the slant-height of the frustum and divide by 2, for the area of the convex surface. Add the areas of the two ends.

To compute the area of the surface of a pyramid.

Rule. Multiply the perimeter of the base by one-half the slant-height and add to the product the area of the base.

To compute the area of the surface of the frustum of a pyramid.

Rule. Multiply the sum of the perimeters of the two ends by the slant-height of the frustum, halve the product, and add to the result the areas of the two ends.

Mensuration of Solids

compute the volume of a prism. (See page 38 for definition of a prism.)

Multiply the area of the base or end by the altitude or perpendicular

This rule applies to prisms with bases or ends of any shape, as long as these or ends are parallel.

To compute the volume of a prismoid.

Definition. A prismoid is a solid with parallel but unequal ends o and with quadrilateral sides.

Rule. To the sum of the areas of the two ends or bases add four ti area of the middle section parallel to them, and multiply this sum by o of the altitude or perpendicular height.

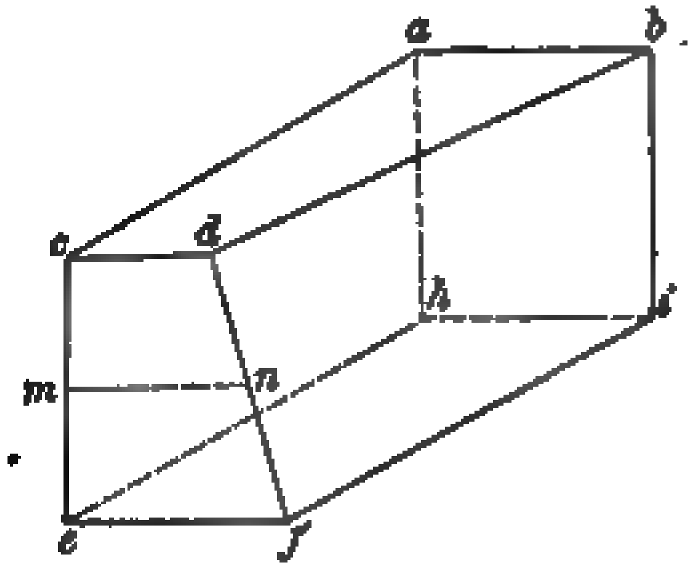


Fig. 38. Quadrangular Prismoid

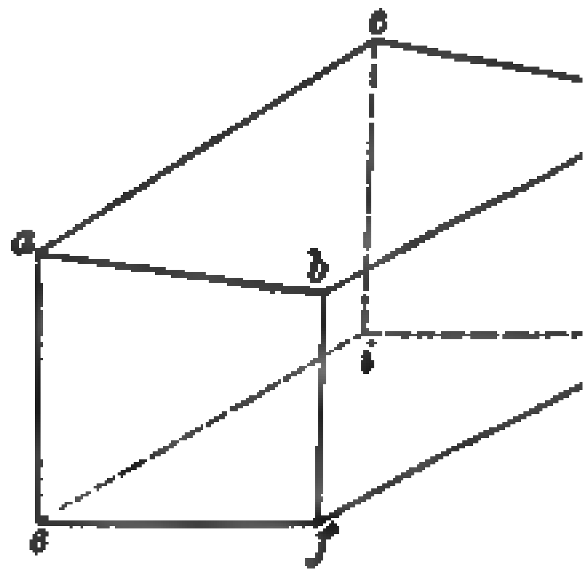


Fig. 39. Prism Truncated Obliq

Example. What is the volume of a quadrangular prismoid, as Fig. which $ab = 6$ in, $cd = 4$ in, $ac = he = 10$ in, $ce = 8$ in, $ef = 8$ in and $ih = 6$

Solution. Area of top $= \frac{6+4}{2} \times 10 = 50$ sq in

Area of bottom $= \frac{8+6}{2} \times 10 = 70$ sq in

Area of middle section $= \frac{6+6}{2} \times 10 = 60$ sq in

$[50 + 70 + (4 \times 60)] \times \frac{4}{3} = 480$ cu in

b

Note. The length of the end (middle section (as at mn, in Fig. $\frac{cd + ef}{2}$

To find the volume of a prism tru obliquely.

Rule. Multiply the area of the b the average height of the edges.

Example. What is the volume truncated prism (Fig. 39) in which in, $fh = 10$ in, $ea = 10$ in, $ci = 12$ in, di in and $fb = 8$ in?

Fig. 40. Wedge or Right Triangular Prism

Solution. Area of base $= 6 \times 10$ sq in

Average height of edges $= \frac{10 + 12 + 8 + 10}{4} = 10$ in

$60 \times 10 = 600$ cu in

compute the volume of a wedge or right triangular prism when the ends are ...lel and equal.

...le. Multiply the area of one end by the length of the wedge.

compute the volume of a wedge when the ends are not parallel.

...le. Add together the lengths of the three edges, *ab*, *cd* and *ef* (Fig. 40); ...iply their sum by the altitude or perpendicular height of the wedge, and ...ly the breadth of the back, and divide the product by 6.

Regular Polyhedrons

...inition. A regular polyhedron is a solid contained within a certain num... ...similar and equal plane faces, all of which are equal regular polygons. ...following is a list of all the regular polyhedrons:

...) The TETRAHEDRON, or pyramid.
...) The HEXAHEDRON, or cube, which has six square faces.
...) The OCTAHEDRON, which has eight triangular faces.
...) The DODECAHEDRON, which has twelve pentagonal faces.
...) The ICOSAHEDRON, which has twenty triangular faces.

compute the volume of a regular polyhedron.

...le 1. **When the radius of the circumscribing sphere is given.** Multiply ...cube of the radius of the sphere by the multiplier opposite to the polyhedron ...lumn 2 of the following table.

...le 2. **When the radius of the inscribed sphere is given.** Multiply the cube ...e radius of the inscribed sphere by the multiplier opposite to the polyhedron ...lumn 3 of the table.

...le 3. **When the area of the surface of the polyhedron is given.** Cube the ...ce given, extract the square root, and multiply the root by the multiplier ...site to the polyhedron in column 4 of the table.

...ble of Factors for Determining the Volumes of Regular Polyhedrons

Figure	1 Number of sides	2 Factor for volume by radius of circumscribing sphere	3 Factor for volume by radius of inscribed circle	4 Factor for volume by surface
...rahedron........	4	0.5132	13.85641	0.0517
...xahedron.........	6	1.5396	8.0000	0.06804
...tahedron.........	8	1.33333	6.9282	0.07311
...decahedron......	12	2.78517	5.55029	0.08169
...sahedron.........	20	2.53615	5.05406	0.0856

compute the volume of a cylinder.

...le. Multiply the area of the base by the altitude or length.

compute the volume of a cone.

...le. Multiply the area of the base by one-third the altitude.

compute the volume of the frustum of a cone (Fig. 41).

...le. Add together the squares of the diameters of the two ends or bases ...the product of the two diameters; multiply this sum by 0.7854, and this ...uct by the altitude, and then divide this last product by 3.

Example. What is the volume of a frustum of a cone 9 in in height, diameter at the base and 3 in in diameter at the top?

Solution. $5^2 + 3^2 = 34$. $3 \times 5 = 15$. $15 + 3$ the sum of the squares of the two diameters ad the product of the diameters of the ends. $49 \times$ $= 38.4846$.

$$\frac{38.4846 \times 9}{3} = 115.4538 \text{ cu in}$$

To compute the volume of a pyramid.

Rule. Multiply the area of the base by the a or perpendicular height, and take one-third product.

To compute the volume of the frustum of a pyra

Rule. Find the height that the pyramid woul the top were put on, and then compute the vol the completed pyramid and the volume of th added; subtract the latter from the former, and the remainder will be the of the frustum.

Fig. 41. Frustum of Cone

To compute the volume of a sphere.

Rule. Multiply the cube of the diameter by 0.5236.

To compute the volume of a segment of a sphere.

Rule 1. To three times the square of the radius of its base add the sq its height; multiply this sum by the height and the product by 0.5236.

Rule 2. From three times the diameter of the sphere subtract twice the height of the segment; multiply this remainder by the square of the height and the product by 0.5236.

Example. The segment of a sphere has a radius, *ac* (Fig 42), of 7 in for its base, and a height, *cb*, of 4 in: what is its volume?

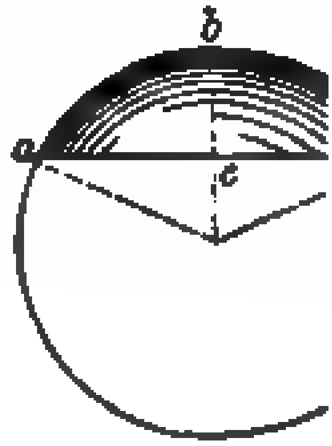


Fig. 42. Segment of

Solution. (By Rule 1) $3 \times 7^2 = 147$, and $147 + 4^2 = 163$, or three times the square of the radius of the base plus the square of the height. $163 \times 4 \times 0.5236 = 341.3872$ cu in = the volume c segment.

Second Solution. By the rule for findi diameter of a circle when a chord and its sine are given, we find that the diameter sphere in this case is 16.25 in, then, by F $(3 \times 16.25) - (2 \times 4) = 40.75$; and $40.75 \times$ $0.5236 = 341.3872$ cu in, the volume c segment.

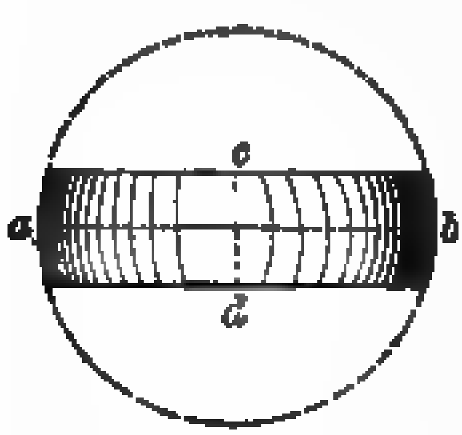


Fig. 43. Zone of Sphere

To compute the volume of a spherical zone.

Definition. The part of a sphere includ tween two parallel planes (Fig. 43).

Rule. To the sum of the squares of the radii of the two ends add one of the square of the height of the zone; multiply this sum by the height an product by 1.5708.

...mpute the volume of a prolate spheroid. (See page 60.)

...le. Multiply the square of the short axis by the long axis and this product ...5236.

...mpute the volume of an oblate spheroid.

...le. Multiply the square of the long axis by the short axis and this product ...5236.

...mpute the volume of a paraboloid of revolution (Fig. 44).

...le. Multiply the area of the base ... the altitude.

...mpute the volume of a hyperbo- ...revolution (Fig. 45).

...le. To the square of the radius ...e base add the square of the ...e diameter; multiply this sum ...e height and the product by ....

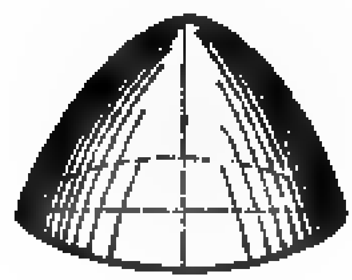


Fig. 44. Paraboloid of Revolution Fig. 45. Hyperboloid of Revolution

...mpute the volume of any figure ...olution.

...le. Multiply the area of the generating surface by the circumference ...ibed by its center of gravity.

...mpute the volume of an excavation, where the ground is irregular and ...ttom of the excavation is level (Fig. 46).

...le. Divide the surface of the ground to be excavated unto equal squares ...out 10 ft on a side, and ascertain by means of a level the height of each ...er, a, a, a, b, b, b, etc., above the level to which the ground is to be excavated. ...n add together the heights of all the corners that come in one square only. Next take twice the sum of the heights of all the corners that come in two squares, as b, b, b; next three times the sum of the heights of all the corners that come in three squares, as c, c, c; and then four times the sum of the heights of all the corners that belong to four squares, as d, d, d, etc. Add together all these quantities, and multiply their sum by one-fourth the area of one of the squares. The result will be the volume of the excavation.

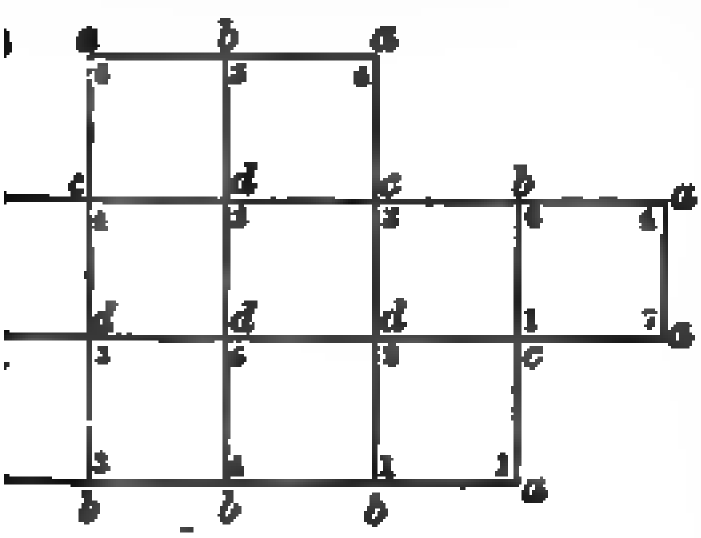


Fig. 46. Plan of Excavation

...mple. Let the plan of an excavation for a cellar be as shown in Fig. 46, ...the heights of each corner above the proposed bottom of the cellar be as ...n by the numbers in the figure. Then the volume of the cellar will be as ...ws, the area of each square being $10 \times 10 = 100$ sq ft:

...lume $= \frac{1}{4}$ of $100\ (a\text{'s} + 2\,b\text{'s} + 3\,c\text{'s} + 4\,d\text{'s})$

The a's in this case	$= 4 + 6 + 3 + 2 + 1 + 7 + 4$	$= 27$
$2 \times$ the sum of the b's	$= 2 \times (3 + 6 + 1 + 4 + 3 + 4)$	$= 42$
$3 \times$ the sum of the c's	$= 3 \times (1 + 3 + 4)$	$= 24$
$4 \times$ the sum of the d's	$= 4 \times (2 + 3 + 6 + 2)$	$= 52$
		145

...lume $= 25 \times 145 = 3\,625$ cu ft, the quantity of earth to be excavated.

4. GEOMETRICAL PROBLEMS

Problem 1. To bisect, or divide into equal parts, a given line, ab (Fig

From *a* and *b*, with any radius greater than half of *ab*, describe ar secting in *c* and *d*. The line *cd*, connecting these intersections, will b and be perpendicular to it.

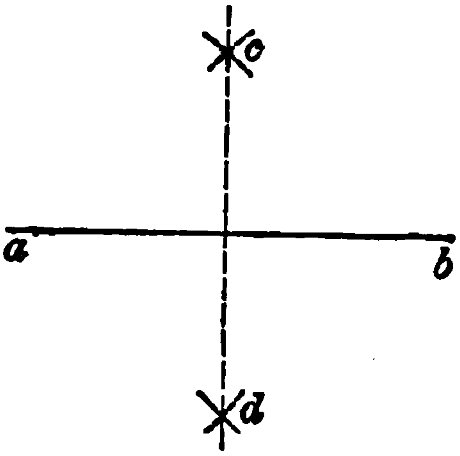


Fig. 47. Line Bisected

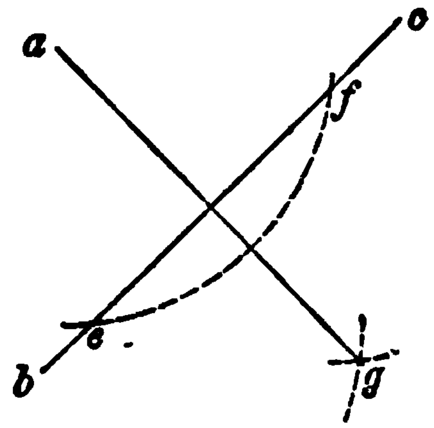


Fig. 48. Perpendicular from Point to Given Line

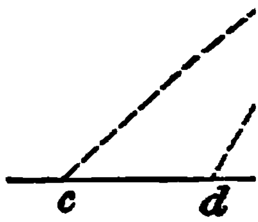


Fig. 49. Perpe from Point to Gi

Problem 2. To draw a perpendicular to a given straight line from a poi out it.

First Method (Fig. 48). From the point *a* describe an arc cutting *bc* in two places, as *e* and *f*. From *e* and *f* describe two arcs, with th radius, intersecting in *g*; then a line drawn from *a* to *g* is perpendicular line *bc*.

Second Method (Fig. 49). From any two points, *d* and *c*, at some d apart in the given line, and with radii *da* and *ca* respectively, descr

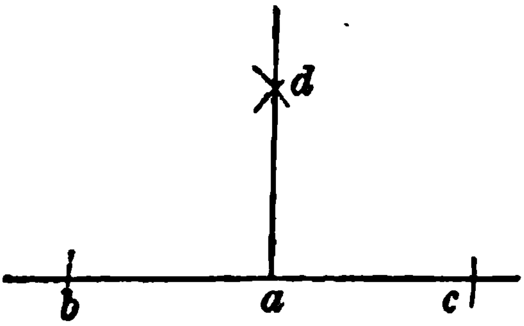


Fig. 50. Perpendicular from Point in Given Line

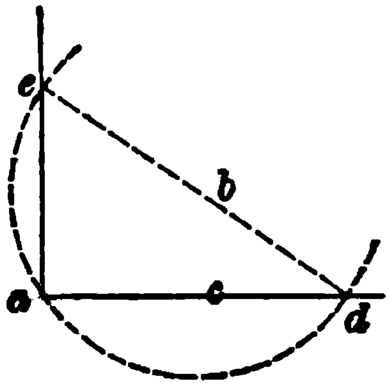


Fig. 51. Perpendicular from Extremity of Given Line

cutting at *a* and *e*. Draw *ae*, which is the perpendicular reequired. method is useful where the given point is opposite the end of the line, or so.

Problem 3. To draw a perpendicular to a straight line from a given p in that line.

First Method (Fig. 50). With any radius, from the given point *a* in th describe arcs cutting the line in the points *b* and *c*. Then with *b* and *c* as c and with any radius greater than *ab* or *ac*, describe arcs cutting each o *d*. The line *da* is the perpendicular required.

...nd Method (Fig. 51), when the given point is at the end of the line. ...e any point, *b*, outside of the line, and with a radius *ba*, describe a semi... passing through *a* and cutting the given line ... Through *b* and *d* draw a straight line inter...ing the semicircle at *e*. The line *ea* will then be ...ndicular to the line *ac* at the point *a*.

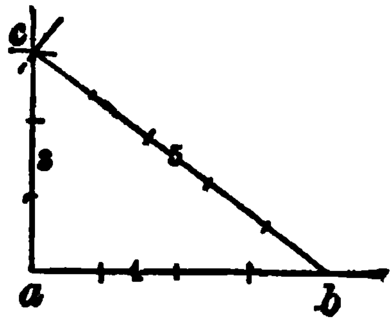
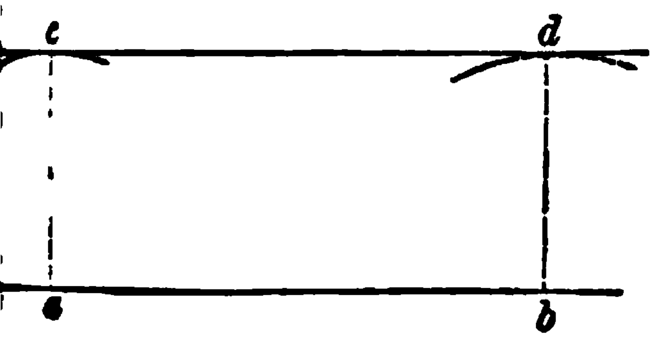


Fig. 52. Perpendicular from Extremity of Given Line

...ird Method (Fig. 52), or the 3, 4 and 5 Method. ...n the point *a* on the given line measure off 4 in, ...ft, or 4 of any other unit and with the same unit ...easure describe an arc, with *a* as a center and ...its as a radius. Then from *b* describe an arc ...a radius of 5 units, cutting the first arc in *c*. ...ne *ca* is the perpendicular required. This method ...rticularly useful in laying out a right angle on ...ground, or framing a house where the foot is used as the unit and the lines ...laid off by the straight-edge.

...laying out a right angle on the ground, the proportions of the triangle may ...30, 40 and 50, or any other multiple of 3, 4 and 5; and it can best be laid ...with the tape. Thus, first measure off, say 40 feet from *a* (Fig. 52) on the ...n line; then let one person hold the end of the tape at *b*, another hold the tape at the 80-ft mark at *a*, and a third person take hold of the tape at the 50-ft mark, with his thumb and finger, and pull the tape taut. The 50-ft mark will then be at the point *c* in the line of the perpendicular.



...g. 53. Straight Line Parallel to Given Line

Problem 4. To draw a straight line parallel to a given line at a given distance away (Fig. 53).

...m any two points near the ends of the given line describe two arcs about ...site the given line. Draw the line *cd* tangent to these arcs and it will be ...llel to *ab*.

...blem 5. To construct an angle equal to a given angle (Fig. 54).

...th the point *A*, at the apex of the given angle, as a center, and any radius, ...ibe the arc *BC*. With the point *a*, at the vertex of the new angle, as a

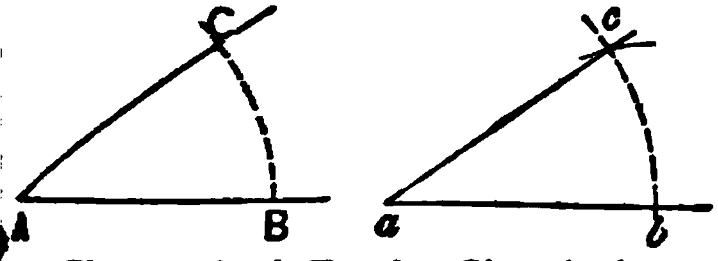


Fig. 54. Angle Equal to Given Angle

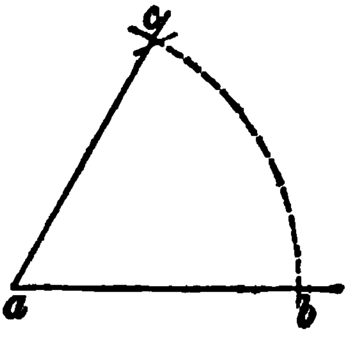


Fig. 55. Angle of 60°

...r, and with the same radius as before, describe an arc, as *BC*. With *BC* ...radius and *b* as a center, describe an arc cutting the other arc at *c*. Then ...cab be equal to the given angle *CAB*.

...blem 6. From a point on a given line to draw a line making an angle of ...th the given line (Fig. 55).

...e any distance, as *ab*, as a radius, and with *a* as a center, describe the arc. ...With *b* as a center and with the same radius, describe an arc cutting the

first one at c. Draw from a a line through c, and it will make with ab a
of 60°.

Problem 7. From a given point, A, on a given line, AE, to draw a line an angle 45° with the given line (Fig. 56).

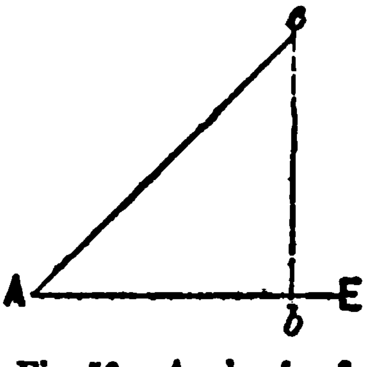


Fig. 56. Angle of 45°

Measure off from A, on AE, any distance, at b draw a line perpendicular to AE. Meas on this perpendicular bc equal to Ab and draw from A through c. This line Ac will make an a 45° with AE.

Problem 8. From any point, A, on a given draw a line which will make any desired angle given line (Fig. 57).

To solve this problem the tables of chords on p to 89 are used. Find in the table the length o to a radius 1, for the given angle. Then ta radius, as large as convenient and describe an arc of a circle bc, with center. Multiply the chord of the angle, found in the table, by the le the radius Ab, and with the product as a new radius and with b as a describe a short arc cutting bc in d. Draw a line from A through d and it will make the required angle with DE.

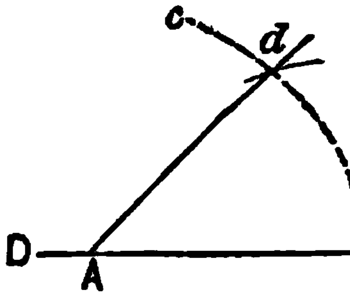


Fig. 57. Line Maki Angle with Given

Example. Draw a line from A on DE, making an angle of 44° 40′ with DE (Fig. 57).

Solution. The largest convenient radius for the arc is 8 in. With A as a center and 8 in as a radius, describe the arc bc. In the table of chords, the chord for an angle or arc of 44° 40′ to a radius 1 is 0.76. Multiplying this by 8 in, the length of the new radius is 6.08 in; and with this as radius and with center, describe an arc cutting bc in d. Ad will be the line required.

Problem 8a. To lay off a given angle approximately, by means of an two-foot rule.

Tables of Angles Corresponding to Openings of a Two-Foot Rule*

In.	Deg.	Min.	In.	Deg.	Min.	In.	Deg.	Min.	In.	Deg.	Min.	In.	De…
¼	1	12	…	11	22	4½	21	37	…	32	3	8¾	
..	1	48	2½	11	58	…	22	13	6¾	32	40	…	
½	2	24	…	12	34	¾	22	50	…	33	17	9	
..	3	00	¾	13	10	…	23	27	7	33	54	…	
¾	3	36	…	13	46	5	24	3	…	34	33	¼	
..	4	11	3	14	22	…	24	39	¼	35	10	…	
1	4	47	…	14	58	¼	25	16	…	35	47	½	
..	5	23	¼	15	34	…	25	53	½	36	25	…	
¼	5	58	…	16	10	½	26	30	…	37	3	¾	
..	6	34	½	16	46	…	27	7	¾	37	41	…	
½	7	10	…	17	22	¾	27	44	…	38	19	10	
..	7	46	¾	17	59	…	28	21	8	38	57	…	
¾	8	22	…	18	35	6	28	58	…	39	35	¼	
…	8	58	4	19	12	…	29	35	¼	40	13	…	
2	9	34	…	19	48	¼	30	11	…	40	51	½	
…	10	10	¼	20	24	…	30	49	½	41	29	…	
¼	10	46	…	21	00	½	31	26	…	42	7	…	

* Trautwine.

Lay one leg of the rule on the paper or board with its inner edge coinciding with the given line. Open the rule until the distance between the inner edges at ends correspond with that given for the angle in the following table; then draw a line by marking along the inner edge of the other leg, and it will give the desired angle within a very close approximation.

Problem 9. To bisect a given angle, as BAC (Fig. 58).

With *A* as a center and any radius, describe an arc, as *cb*. With *c* and *b* as centers, and any radius greater than one-half of *cb*, describe two arcs, intersecting at *d*. Draw from *A* a line through *d* and it will bisect the angle *BAC*.

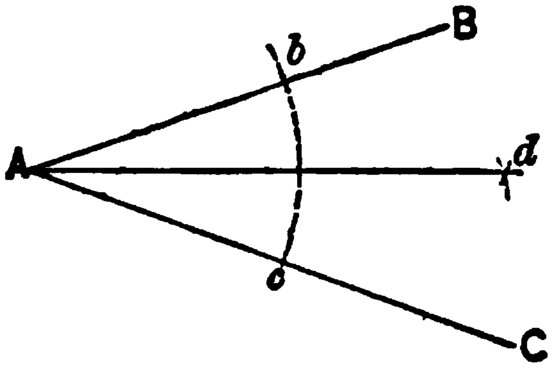


Fig. 58. Angle Bisected

Problem 10. To bisect the angle included between two lines, as AB and CD, when the vertex of the angle is not on the drawing (Fig. 59).

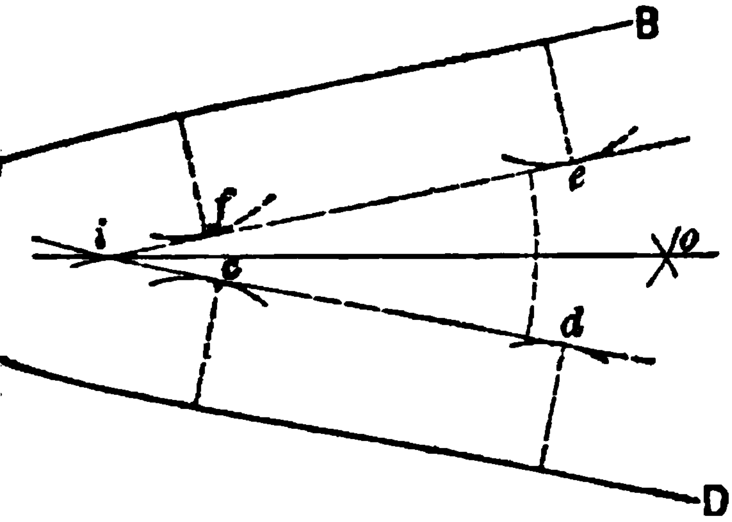


Fig. 59. Angle Bisected. Angle not on Drawing

Draw *fe* parallel to *AB* and *cd* parallel to *CD*, so that the two lines intersect, as at *i*. Bisect the angle *eid*, as in the preceding problem, and draw a line through *i* and *o* which will bisect the angle between the two given lines.

Problem 11. Through two given points, B and C, to describe an arc of a circle with a given radius (Fig. 60).

With *B* and *C* as centers and with a radius equal to the given radius, describe two arcs intersecting at *A*. With *A* as a center and the same radius, describe the arc *bc*, which will pass through the given points, *B* and *C*.

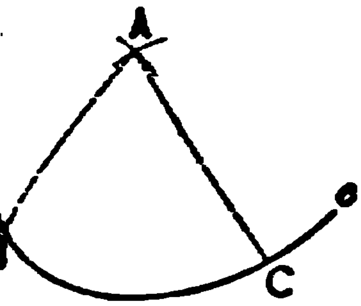


Fig. 60. Circular Arc Through Two Given Points

Problem 12. To find the center of a given circle (Fig. 61).

Draw any chord in the circle, as *ab*, and bisect this chord by the perpendicular *cd*. This line will pass through the center of the circle and *ef* will be a diameter of the circle. Bisect *ef*, and the center *o* will be the center of the circle.

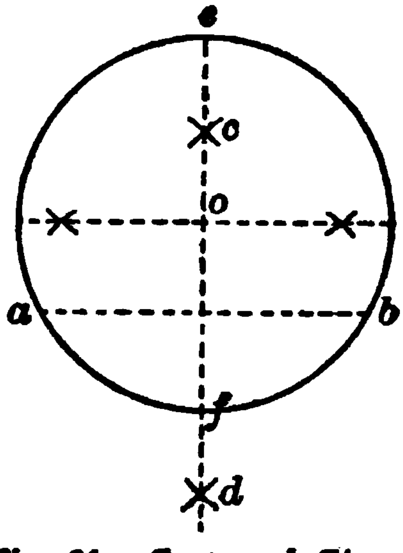


Fig. 61. Center of Given Circle

Problem 13. To draw a circular arc through three given points, as A, B and C (Fig. 62).

Draw lines from *A* to *B* and from *B* to *C*. Bisect *AB* and *BC* by the lines *aa* and *cc* and prolong these lines until they intersect at *o*, which will be the center for the arc sought. With *o* as a center and *Ao* as radius, describe the arc *ABC*.

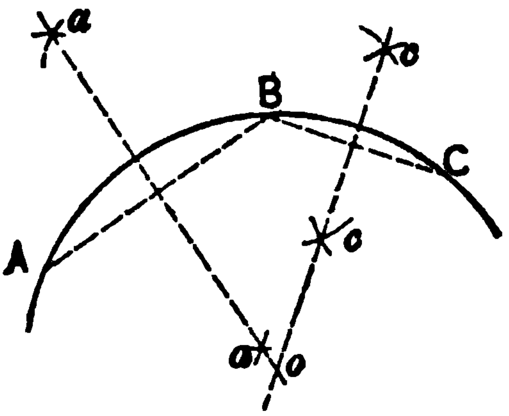


Fig. 62. Circular Arc Through Three Given Points

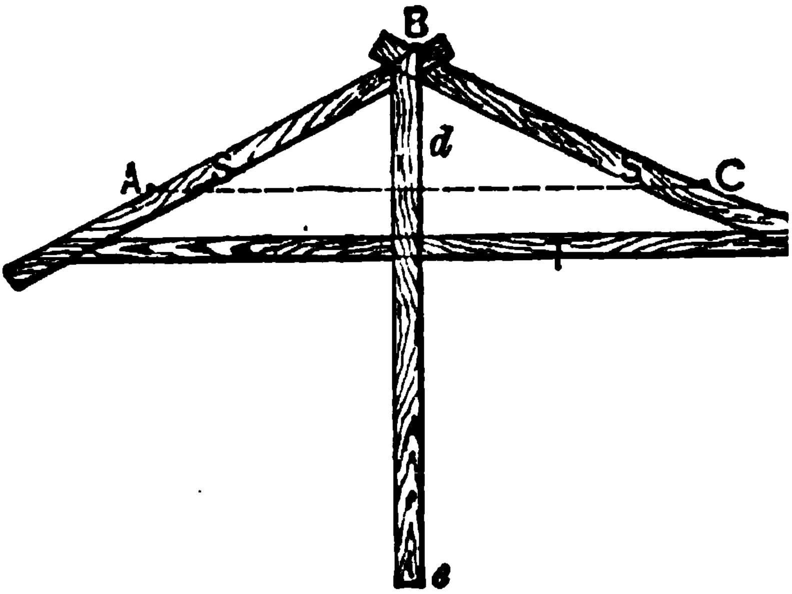


Fig. 63. Frame for Drawing Circular Ar

Problem 14. To describe a circular arc passing through three given when the center is not available, by means of a triangle (Fig. 63).

Let *A*, *B* and *C* be the given points. Insert two stiff pins or nails at *A* a Place two strips of wood, *SS*, as shown in the figure, one against *A*, the against *C*, and inclined so that their intersection shall come at the third *B*. Fasten the strips together at their intersection and nail a third stri to their other ends, so as to make a firm triangle. Place the pencil-point and, keeping the edges of the triangle against *A* and *C*, move the triangle t left and right. The pencil-point will describe the required arc.

When the points *A* and *C* are at the same distance from *B*, if a strip of is nailed to the triangle, so that its edge *de* is at right-angles to a line jo *A* and *C*, as the triangle is moved one way or the other, the edge *de* will al point to the center of the circle. This principle is used in linear perspecti

Problem 15. To describe a circular arc which will be tangent at a given A, to a straight line, and pass through a given point, C, outside the line (Fig

Draw from *A* a line perpendicular to the given line. Connect *A* and *C* straight line and bisect this line by the perpendicular *ac*. The point where two perpendiculars intersect is the center of the circle.

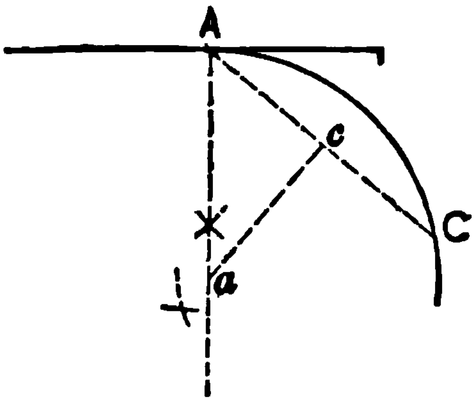


Fig. 64. Circular Arc Tangent to Line at Given Point

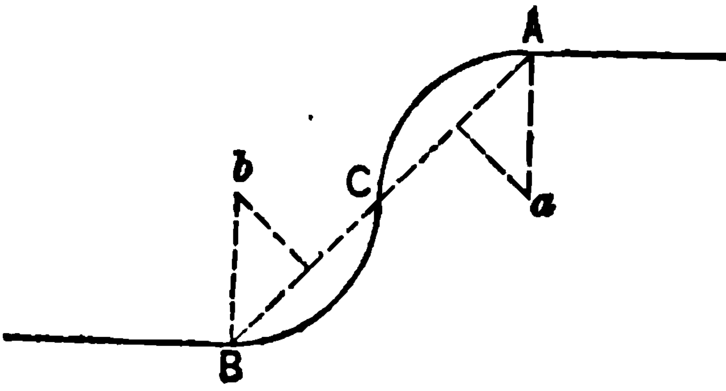


Fig. 65. Reversed Curve Between Parallel

Problem 16. To connect two parallel lines by a reversed curve compos two circular arcs of equal radius, and tangent to the lines at given points, and B (Fig. 65).

Join *A* and *B* and divide the line into two equal parts at *C*. Bisect *CA* *CB* by perpendiculars. At *A* and *B* erect perpendiculars to the given lines the intersections *a* and *b* will be the centers of the arcs composing the req curve.

[...]lem 17. On a given line, as AB (Fig. 66), to construct a compound curve [...]ree arcs of circles, the radii of the two side arcs being equal and of a given [...]h, and their centers [...]g in the given line. The [...]al arc is to pass through [...]en point, C, on the perpendicular bisecting the given [...], and is to be tangent to [...]other two arcs.

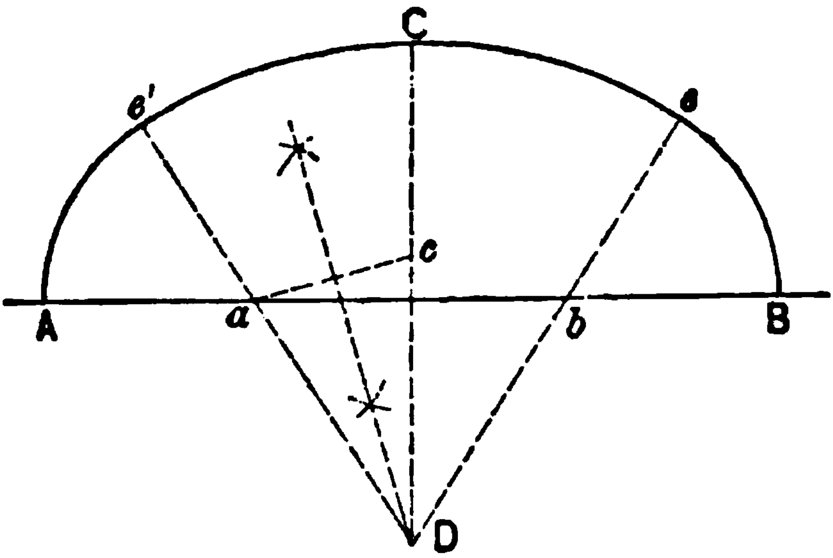
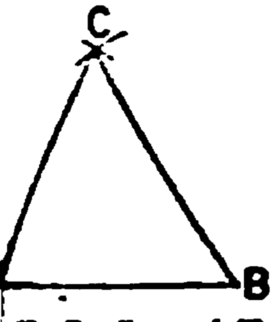


Fig. 66. Curve of Three Circular Arcs

[...]raw the perpendicular [...]. Lay off Aa, Bb and Cc, [...]h equal to the given radius [...] the side arcs; draw ac; [...]ct ac by a perpendicular. [...]e intersection of this line [...]h the perpendicular CD [...]e required center of the [...]tral arc. Through a and b draw the lines Dc and De'; from a and b, with [...] given radius, equal to Aa, Bb, describe the arcs Ae' and Be; from D as a center, and with CD as a radius, describe the arc eCe' which completes the curve required.



[...]. 67. Equilateral Triangle on Given Base

Problem 18. To construct a triangle upon a given straight line or base, the length of the two sides being given (Figs. 67 and 68).

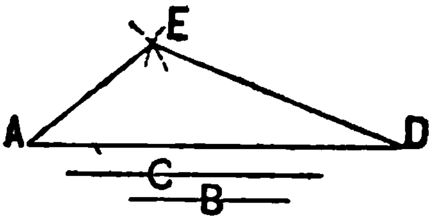
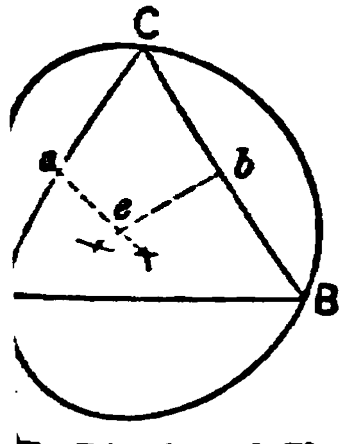


Fig. 68. Scalene Triangle on Given Base

First. An equilateral triangle (Fig. 67). With the [...]mities A and B of the given line as centers and with AB as a radius. [...]ibe arcs cutting each other at C. Join AC and BC.

[...]cond. A scalene triangle (Fig. 68). Let AD be the given base and the other [...] sides be equal to C and B. With D as a center, and with a radius equal to C, describe at E an arc of indefinite length. With A as a center and with B as a radius, describe an arc cutting the first at E. Join E with A and D. ADE is the required triangle.



[...]. Triangle and Circumscribed Circle

Problem 19. To describe a circle about a triangle (Fig. 69).

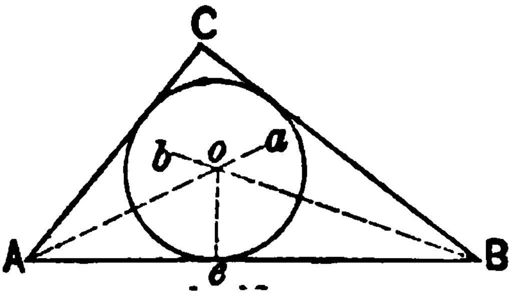


Fig. 70. Triangle and Inscribed Circle

Bisect two of the sides, as AC and CB, of the triangle, and at their centers, erect perpendicular lines, [...]d be, intersecting at e. With e as a center, and eC as a radius, describe [...]. It will pass through A and B.

[...]em 20. To inscribe a circle in a triangle (Fig. 70).

[...]t two of the angles, A and B, of the triangle by lines cutting each other [...] With o as a center, and with oe as a radius, describe a circle. It will be [...]t to the other two sides.

Problem 21. To inscribe a square in a circle and to describe a circle ab square (Fig. 71).

To inscribe the square. Draw two diameters, AB and CD, at right-a to each other. Join the points A, D, B and C. $ADBC$ is the inscribed sq

To describe the circle. Draw the diagonals as before, intersecting at E, with E as a center and AE as a radius, describe the circle.

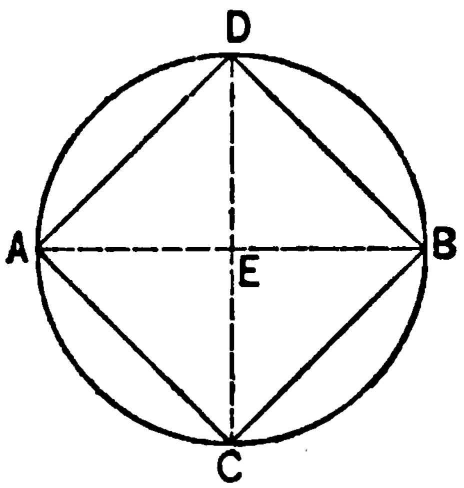


Fig. 71. Inscribed Square and Circumscribed Circle

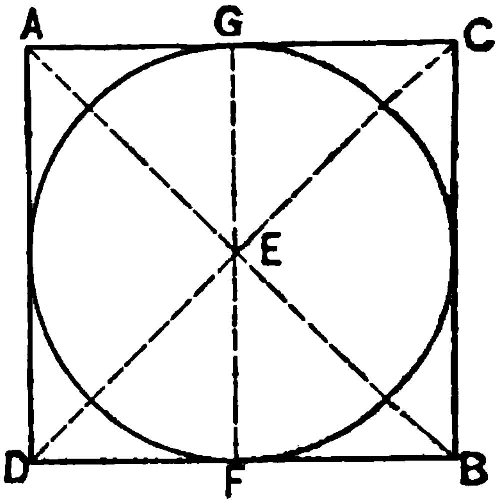


Fig. 72. Inscribed Circle and Circumscribed Square

Problem 22. To inscribe a circle in a square and to describe a square ab circle (Fig. 72).

To inscribe the circle. Draw the diagonals AB and CD, intersecting Draw the perpendicular EG to one of the sides. Then with E as a center EG as a radius, describe a circle. It will be tangent to all four sides o square.

To describe the square. Draw two diameters, AB and CD, at right- to each other, and prolonged beyond the circumference. Draw the dia GF, bisecting the angle CEA or BED. Draw lines through G and F pe dicular to GF, and terminating in the diagonals. Draw AD and CB to plete the square.

Problem 23. To inscribe a pentagon in a circle (Fig. 73).

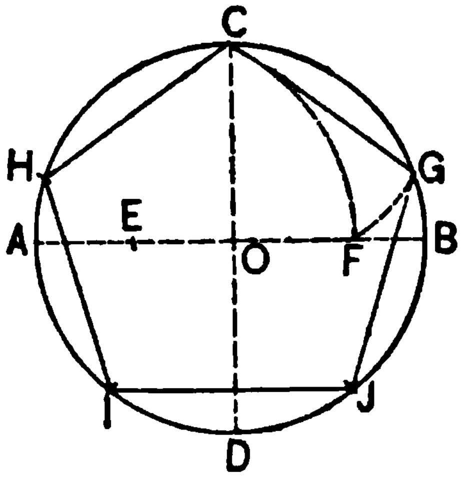


Fig. 73. Circle and Inscribed Pentagon

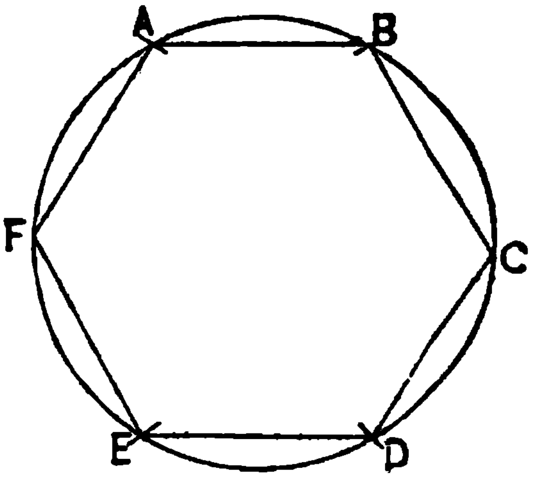


Fig. 74. Circle and Inscribed Hexagon

Draw two diameters, AB and CD, at right-angles to each other. Bisec at E. With E as a center and EC as a radius, cut OB at F. With C as a c and CF as a radius, cut the circle at G and H. With these points as center the same radius, cut the circle at I and J. Join I, J, G, C and H. $IJGC$ the inscribed regular pentagon.

Problem 24. To inscribe a regular hexagon in a circle (Fig. 74).

Lay off on the circumference the radius of the circle six times, and connect the points.

Problem 25. To construct a regular hexagon upon a given straight line, AB (Fig. 75).

From *A* and *B*, with a radius equal to *AB*, describe arcs intersecting at *O*. With *O* as a center and a radius equal to *AB*, describe a circle, and from *A* or *B* lay off the lengths *BC*, *CD*, *DE*, *EF* and *FA* on the circumference of the circle. *ABCDEFA* is the required regular hexagon.

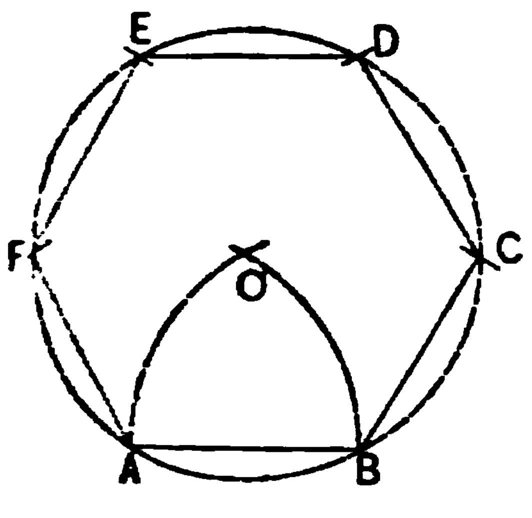


Fig. 75. Regular Hexagon on Given Line

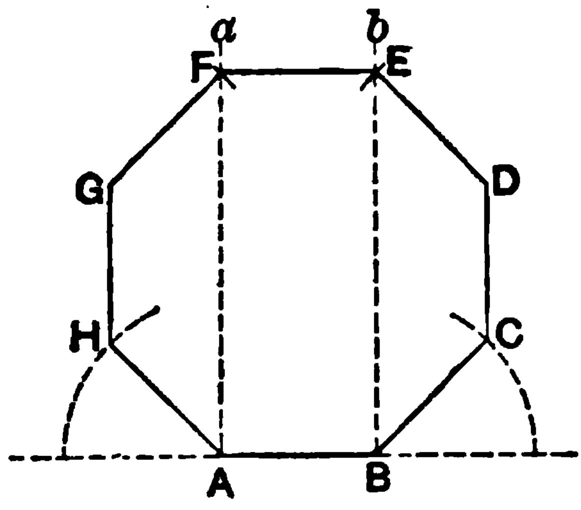


Fig. 76. Regular Octagon on Given Line

Problem 26. To construct a regular octagon upon a given straight line, AB (Fig. 76).

Produce the line *AB* both ways and draw the perpendiculars *Aa* and *Bb*, of indefinite length. Bisect the external angles at *A* and *B* and make the length of the bisecting lines equal to *AB*. From *H* and *C* draw lines parallel to *Aa* or *Bb* and equal in length to *AB*. From *G* and *D* as centers describe arcs, with radius *AB*, cutting the perpendiculars *Aa* and *Bb* in *F* and *E*. Draw *GF*, *FE* and *ED*. *ABCDEFGHA* is the required octagon.

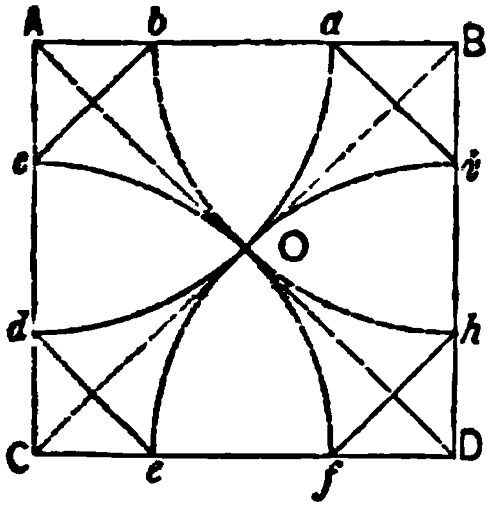


Fig. 77. Square and Inscribed Regular Octagon

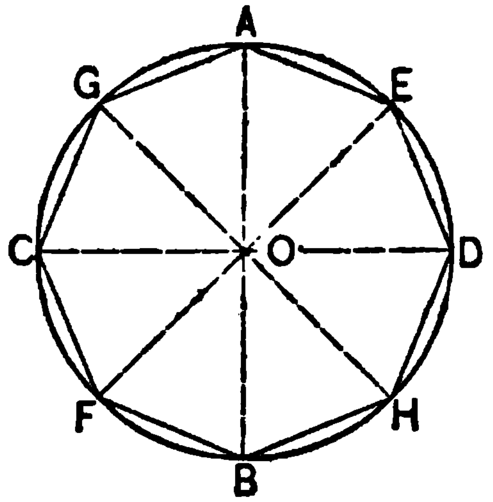


Fig. 78. Circle and Inscribed Regular Octagon

Problem 27. To construct a regular octagon in a square (Fig. 77).

Draw the diagonals *AD* and *BC* and from *A*, *B*, *C* and *D*, with a radius equal to *AO*, describe arcs cutting the sides of the square in *a*, *b*, *c*, *d*, *e*, *f*, *h* and *i*. Draw *ai*, *hf*, *ed* and *cb*. *aihfedcba* is the required octagon.

Problem 28. To inscribe a regular octagon in a circle (Fig. 78).

Draw two diameters, AB and CD, at right-angles to each other. Bisect angles AOD and AOC by the diameters EF and GH. $AEDHBFCGA$ is required octagon.

Problem 29. To inscribe a circle within a regular polygon.

First. When the polygon has an even number of sides, as in Fig. 79. B two opposite sides at A and B, draw AB and bisect it at C by a diagonal, connecting two opposite angles, as D and E. The circle drawn with a ra and with C as a center is the inscribed circle required.

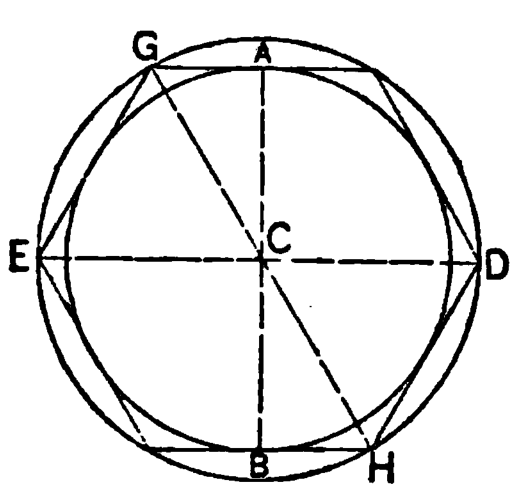


Fig. 79. Regular Polygon, Even Number of Sides, with Inscribed and Circumscribed Circles

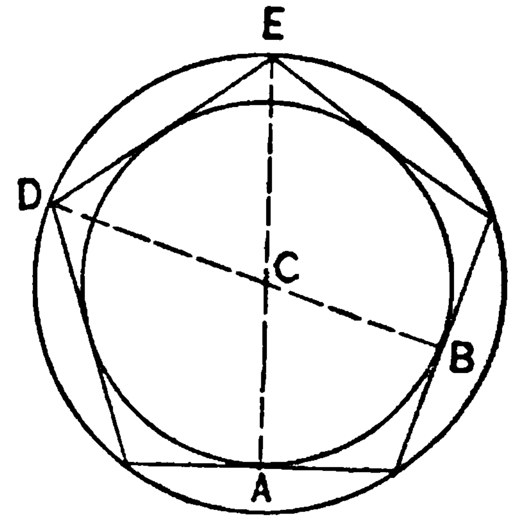


Fig. 80. Regular Polygon. Odd Number of Sides, with Inscribed and Circumscribed Circles

Second. When the number of sides is odd, as in Fig. 80. Bisect two of adjacent sides as at A and B, and draw lines, AE and BD, to the opposite an and intersecting at C. The circle drawn with C as a center and CA as a ra is the inscribed circle required.

Problem 30. To draw a circumscribing circle around a regular polygon.

First. When the number of sides is even, as in Fig. 79. Draw two diago from opposite angles, as ED and GH, intersecting at C. The circle drawn C as a center and with CD as a radius is the circumscribing circle required.

Second. When the number of sides is odd, as in Fig. 80. Determine center, C, as in the last problem. The circle drawn with C as a center and as a radius, is the circumscribing circle required.

Problems on the Ellipse, the Parabola, the Hyperbola and the Cycloid

The Ellipse

Problem 31. To describe an ellipse, the length and breadth, or the two a being given.

First Method (Fig. 81), the two axes, AB and CD, being given. On AB CD as diameters and from the same center, O, describe the circles $AGBH$ $CLDK$. Take any convenient number of points on the circumference of outer circle, as b, b', b'', etc., and from them draw lines to the center, O, cutt the inner circle at the points a, a', a'', etc., respectively. From the poi b, b', etc., draw lines parallel to the shorter axis CD; and from the point

d, etc., draw lines parallel to the longer axis AB, and intersecting the first set of lines at c, c', c'', etc. These last points will be points in the ellipse, and by determining a sufficient number of them, the ellipse can be drawn.

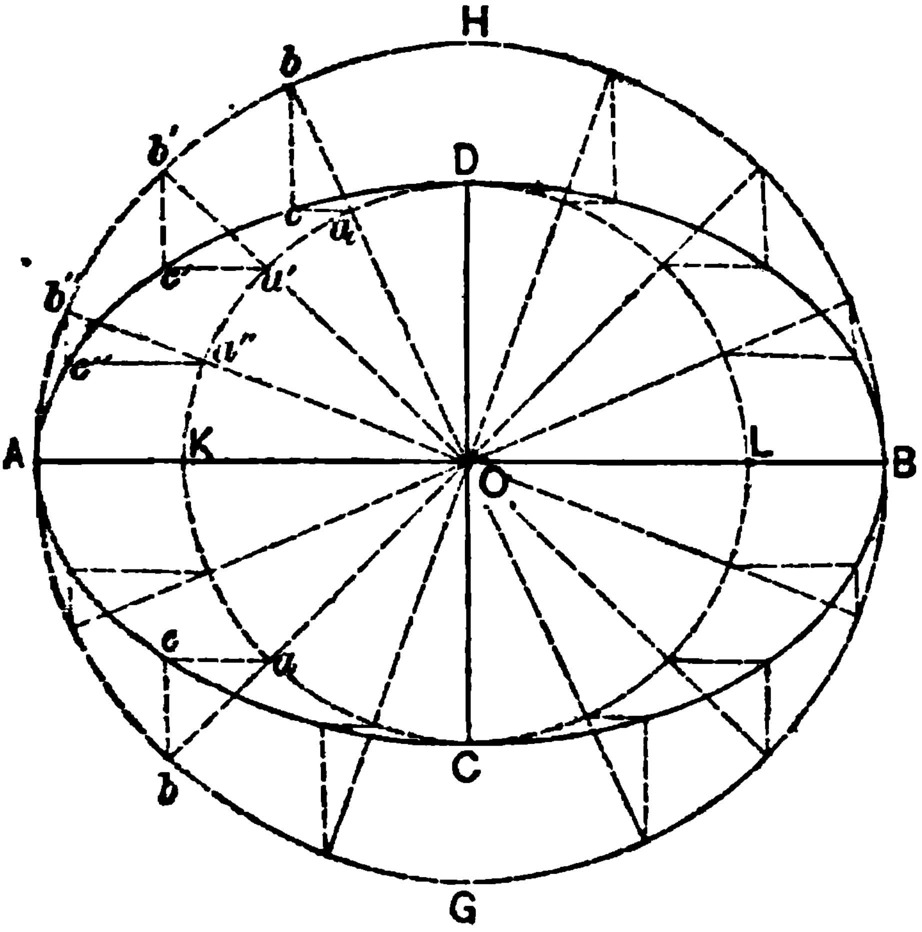


Fig. 81. Ellipse Described on Given Axes.

Second Method (Fig. 82). Take the straight-edge, made of a stiff piece of paper, cardboard or wood, and from some point as a, mark off ab equal to half the shorter diameter CD, and ac equal to half the longer diameter AB. Place the straight-edge so that the point b is on the longer and the point c on the shorter diameter. Then will the point a be over a point in the ellipse. Make on the paper a dot at a and move the straight-edge around, always keeping the points b and c over the major and minor axes respectively. In this way any number of points in the ellipse may be determined and the ellipse drawn.

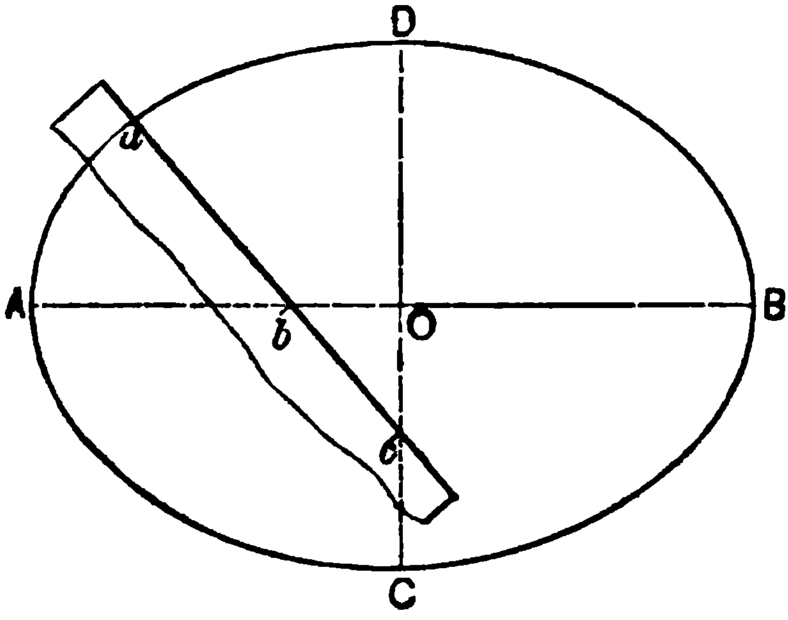


Fig. 82. Ellipse Described with Straight-Edge

Third Method (Fig. 83). Given, the two axes, AB and CD. From the point D as a center, and a radius AO, equal to one-half of AB, describe an arc cutting AB at F and F'. These two points are called the foci of the ellipse.

Note. One property of the ellipse is, that the sums of the distances of any two points on the circumference from the foci are the same. Thus $F'D + DF =$ [illegible] or $F'G + GF$.

Fix two pins in the axis AB at F and F' and loop upon them a thread, or cord equal in length, when fastened to the pins, to AB, so as, when stretched as per dotted line FDF', will just reach to the extremity D of the short axis. Place a pencil-point inside the cord, as at E, and move the pencil along, keeping the cord stretched tight. The pencil-point will trace the ellipse required.

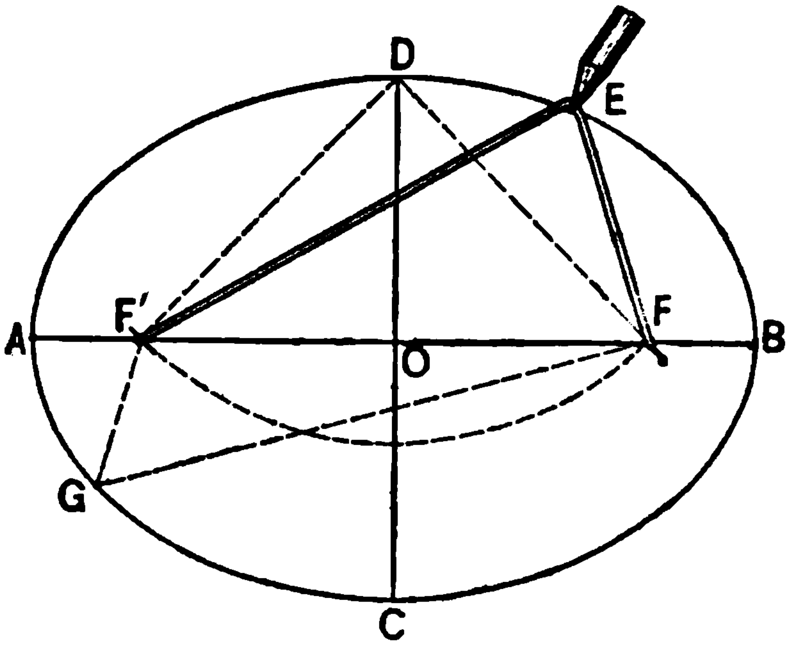


Fig. 83. Ellipse Described with String and Pencil

Problem 32. To draw a tangent to an ellipse at a given point on the curve (Fig. 84).

Let it be required to draw a tangent at the point E on the ellipse shown. First determine the foci F and F' as in the third method for describing an ellipse, and from E draw lines EF and EF'. Prolong EF' to a, so that Ea equals EF. Bisect the angle aEF by describing arcs from a and F as centers, as shown at b, and through b draw a line through E. This line is the tangent required. If it is required to draw a line normal to the curve at E, as, for instance, the joint of an elliptical

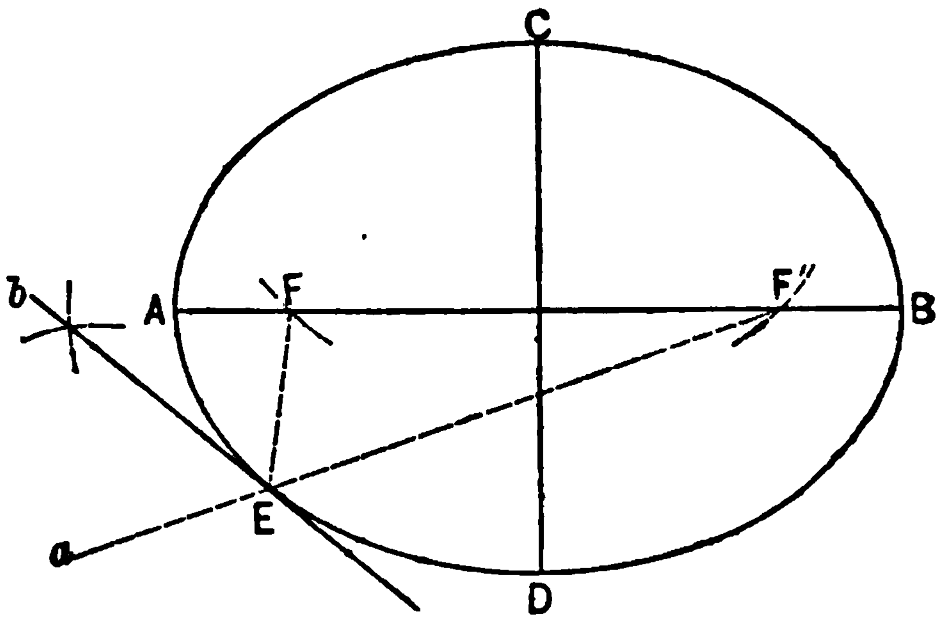


Fig. 84. Tangent Drawn to Point on Ellipse

arch, bisect the angle FEF', and draw the bisecting line through E, and it will be the normal to the curve and the proper line at that point for the joint of the elliptical arch.

Problem 33. To draw a tangent to an ellipse from a given point outside of the curve (Fig. 85).

From the given point T as a center, and with a radius equal to the distance to the nearer focus F, describe an arc of a circle. From F' as a center, and with a radius equal to the length of the longer axis of the ellipse, describe an arc cutting the circle just described at a and b. Draw lines from F' to a and b cutting the ellipse at E and G. Draw lines from T through E and G and these will be the tangents required.

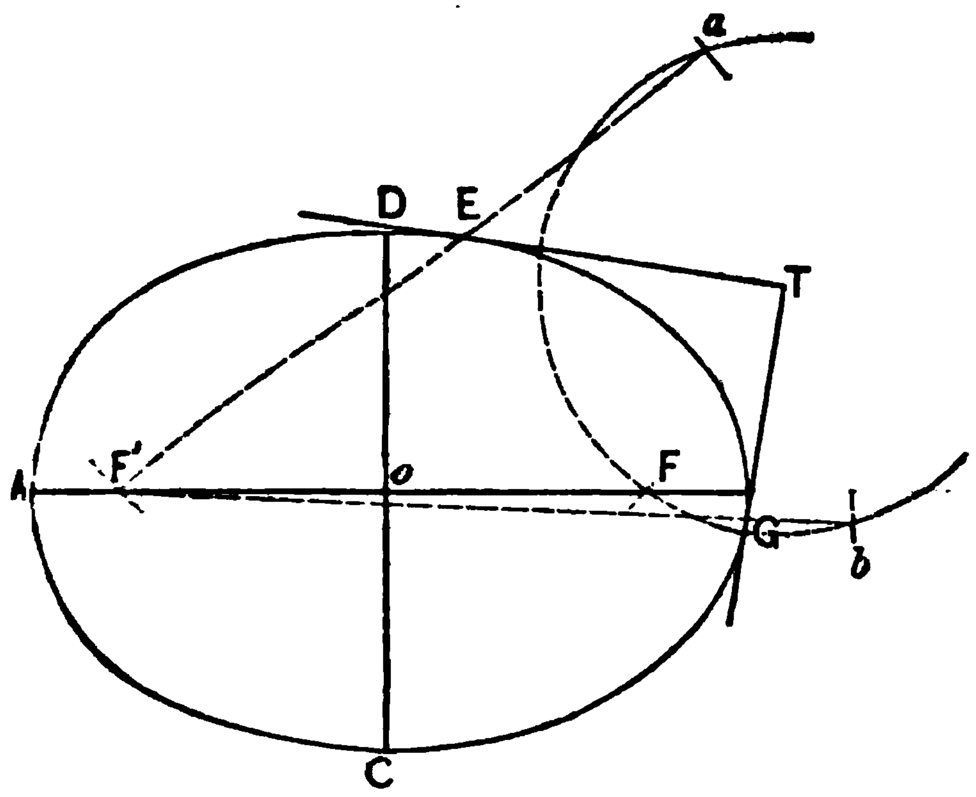


Fig. 85. Tangent Drawn to Ellipse from Point Outside

34. To describe an ellipse approximately, by means of circular arcs.
With arcs of two radii (Fig. 86). Take half the difference of the two
AB and *CD*, and set it off from the center *O* to *a* and *c* on *OA* and *OC*;
and on *AB* set off half *ac* from *a* to *d*; draw *di* parallel to *ac*; set off *Oe* equal
; join *ei* and draw *em* and *dm* parallel respectively to *id* and *ie*. With

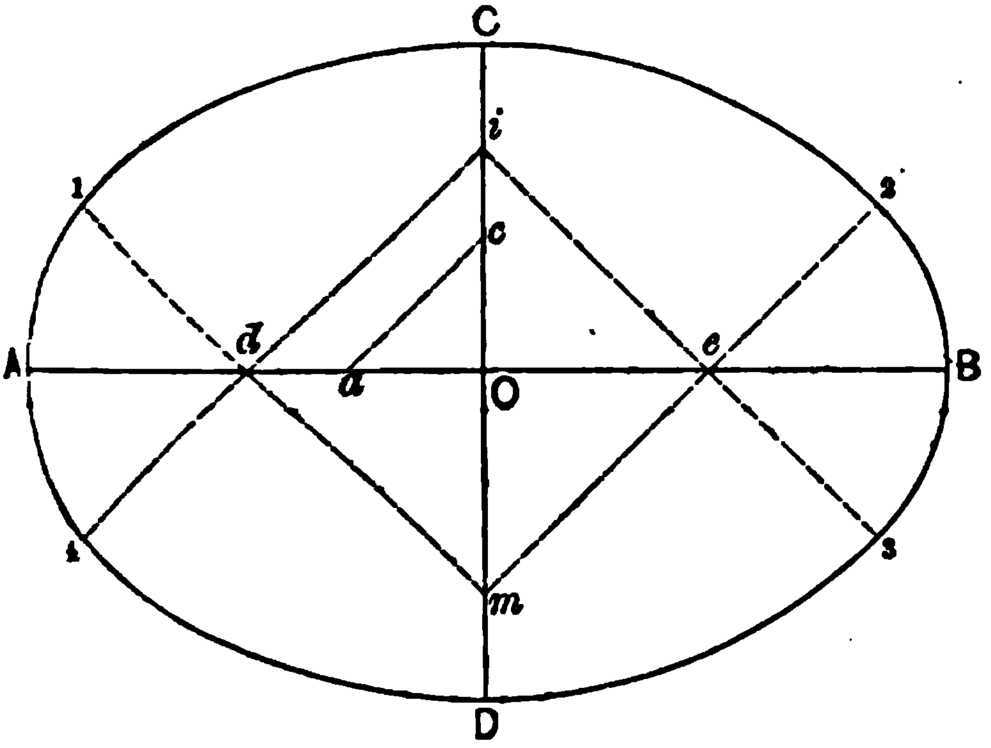


Fig. 86. Ellipse Described with Circular Arcs of Two Radii

a center and with a radius *mC*, describe an arc through *C*, terminating in
1 and 2 on *md* and *me* produced. With *i* as a center, and with *iD*
describe an arc through *D*, terminating in points 3 and 4 on *ie* and
With *d* and *e* as centers, describe arcs through *A* and *B*, connecting
1 and 4 and 2 and 3. The four arcs thus described form approximately
This method is not satisfactory when the conjugate or minor axis
than two-thirds the transverse or major axis.

Another method of approximating an ellipse by means of arcs of two
is shown in Fig. 87, the axis major AB and the semiminor axis OC

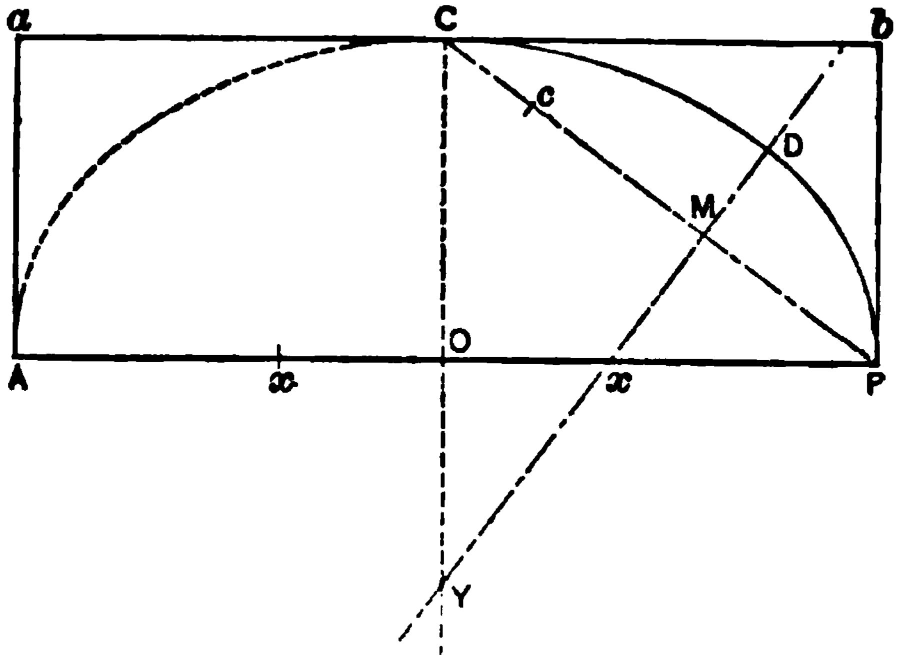


Fig. 87. Ellipse Described with Circular Arcs of Two Radii

given. Draw the
tangle $AabBA$, an
diagonal CB. La
Cc equal to the
ence between OB
OC. Bisect cB at
erect the perpend
YD, intersecting C
duced at Y and
at x. Make Ox'
Then will x, x',
be the three cente
quired, the curve
coming tangent
and at the correspo
point on the left
side of the ellipse.
method results
curve which is s
fuller at the haunches than the curve drawn by the preceding method

Second. With arcs of three radii (Fig. 88). On the transverse or majo

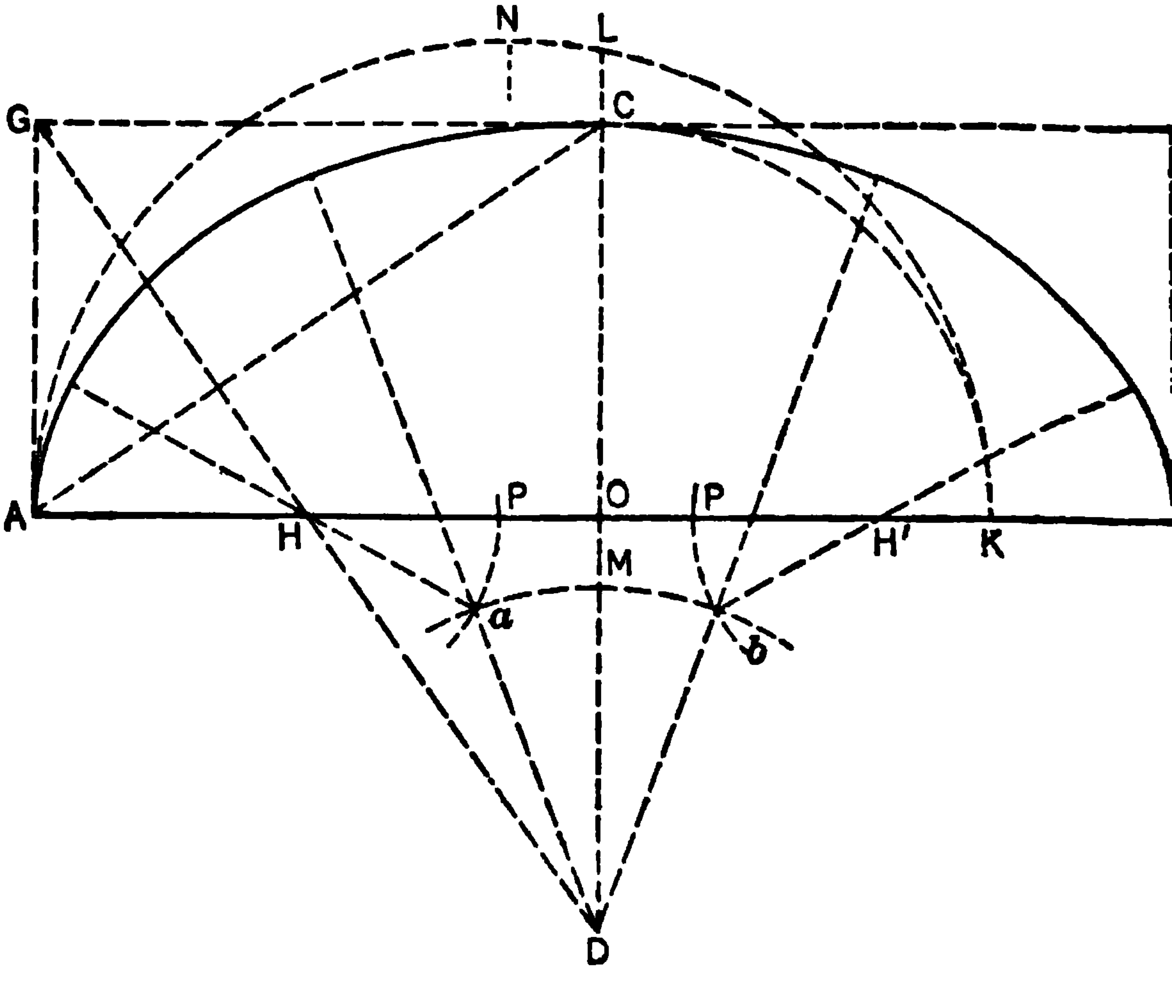


Fig. 88. Ellipse Described with Circular Arcs of Three Radii

AB draw the rectangle $AGEBA$, equal in height to OC, half the conju
minor axis. Draw AC and draw GD perpendicular to AC. Set off OK e

on *AK* as a diameter describe the semicircle *ANK*. Extend *OC* to *D*. Set off *OM* equal to *CL*, and with *D* as a center and with a radius describe an arc. With *A* and *B* as centers and with a radius *OL*, cut *AB* at *P*. From *H* as a center, and with a radius *HP*, cut the arc *ab* at *a*. *b* are determined in like manner. The points *H*, *a*, *D*, *b* and *H'*, are centers of the arcs required.

...the lines *aH*, *Da*, *Db*, and *bH'*, and thus determine the lengths of ... This method is practicable for all ellipses. It is often employed ..., stone arches and bridges.

The Parabola

...em 35. To describe a parabola when the vertex A, the axis AB and a ..., II, of the curve are given (Fig. 89).

...struct the rectangle *ABMCA*. Divide *MC* into any number of equal ..., four for instance. Divide *AC* in like manner. Connect *A*1, *A*2 and *A*3. ...gh 1', 2', 3', draw parallels to the axis *AB*. The intersections I, II and ... of these lines, are points in the required curve.

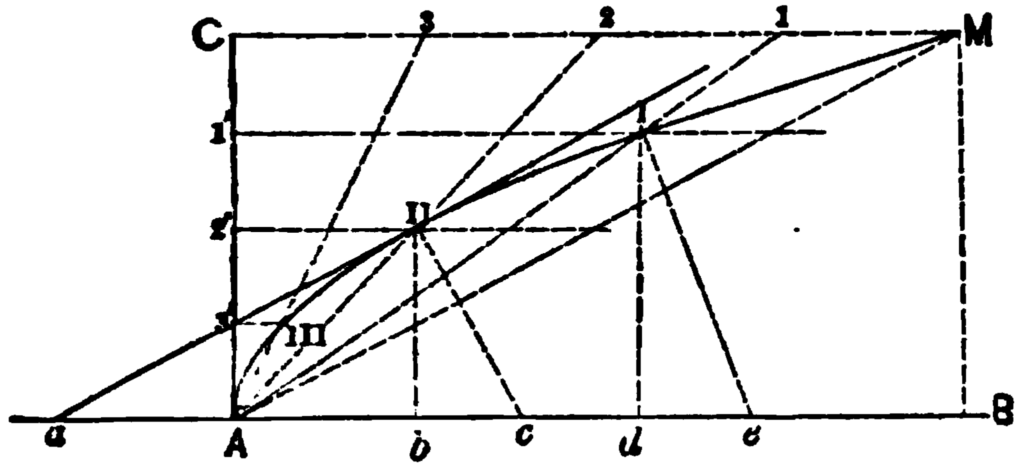


Fig. 89. Parabola and Tangent to Point on Parabola

...lem 36. To draw a tangent to a given point, II, of the parabola (Fig. 89).

...m the given point II let fall a perpendicular on the axis *AB* at *b*. Produce ...is to the left of *A*. Make *Aa* equal to *Ab*. A line drawn through *a* and ... the tangent required. The lines perpendicular to the tangent are called ...als.

...draw a normal to any point, as I, the tangent to any other point, II being ...

...w the normal II*c*. From I, let fall a perpendicular I*d*, on the axis *AB*. ...f *de* equal to *bc*. The line I*e* is the normal required. The tangent may be ...n at I by laying off a perpendicular to the normal I*e* at I.

The Hyperbola

...from any point, *P*, of an hyperbola, two straight lines are drawn to two ... points, as *F* and *F'*, the foci of the hyperbola, their DIFFERENCE is always ...same.

...lem 37. To describe an hyperbola when a vertex, a, the given difference ...d one of the foci, F are given (Fig. 90).

...w the axis *AB* of the hyperbola, with the given distance *ab* and the focus

F marked on it. From *b* lay off bF_1 equal to aF to determine the other fo

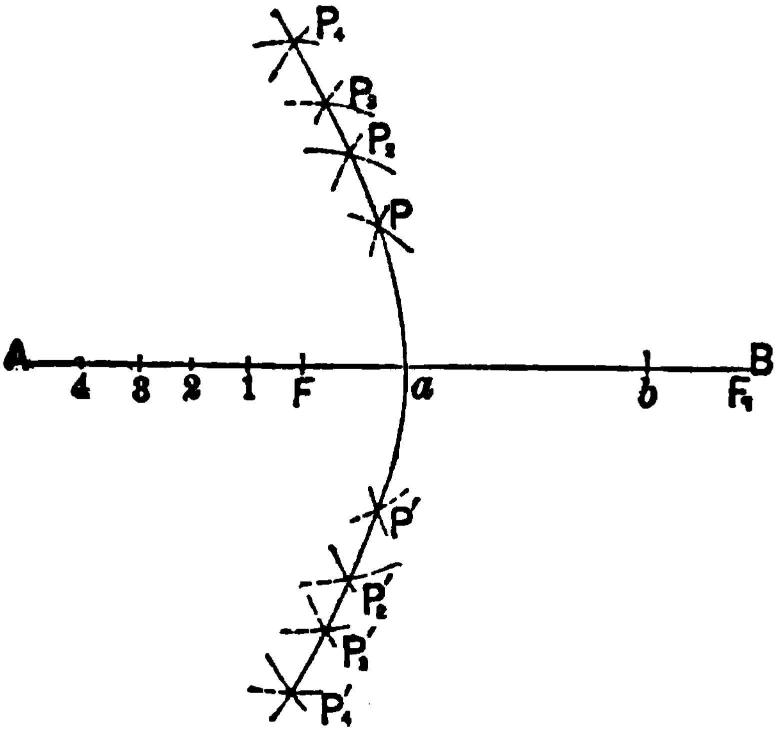


Fig. 90. Hyperbola Described

Take any point, as 1 and with $a1$ as a radius as a center, describe tw arcs above and below t With $b1$ as a radius, an a center, describe arcs those just described, at P'. Take several point 3 and 4, and determine t responding points P_2, P_4 in the same way curve passing through points is an hyperbola.

To draw a tangent point of an hyperbola, lines from the given p each of the foci and the angle thus formed bisecting line is the t required.

The Cycloid

The CYCLOID is the curve described by a point on the circumference of rolling in a straight line.

Problem 38. To describe a cycloid (Fig. 91).

Draw the straight line *AB*. Describe the generating circle tangent line at its middle point *D*, and through the center *C*, of the circle, draw t

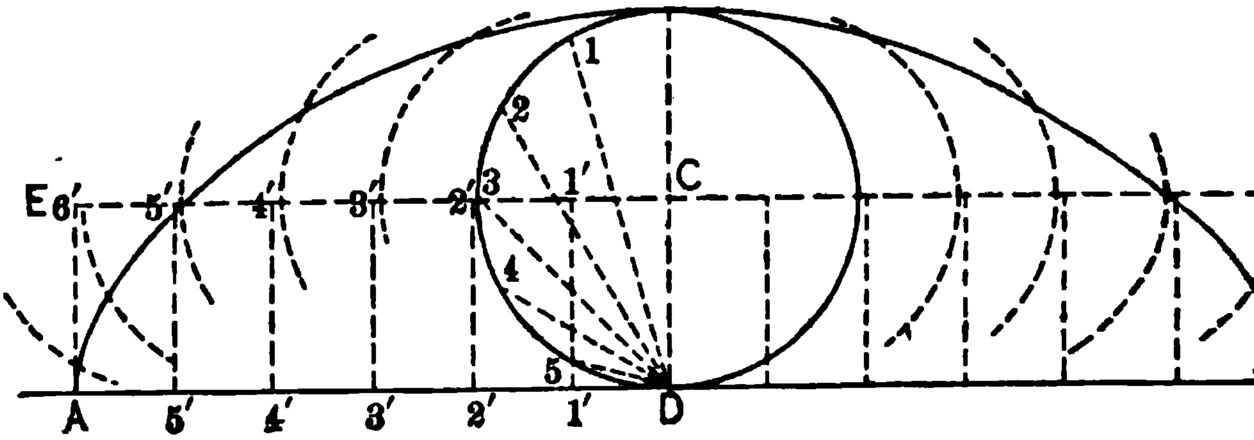


Fig. 91. Cycloid Described

EE parallel to *AB*. Let fall a perpendicular from *C* upon *AB*. Divide the circumference into any number of equal parts, for example, six. Lay off o and *CE* distances $C1'$, $1'2'$, etc., equal to the divisions of the circumfe Draw the chords $D1$, $D2$, etc. From the points $1'$, $2'$, $3'$, etc., on the lin with radii equal to the generating circle, describe arcs as shown. From points $1'$, $2'$, $3'$, $4'$, $5'$, etc., on the line *BA*, and with radii equal respectiv the chords $D1$, $D2$, $D3$, $D4$, $D5$, describe arcs cutting the preceding The intersections are points of the required cycloid.

Table of Chords. Radius = 1.0000

′	0°	1°	2°	3°	4°	5°	6°	7°	8°	9°	10°	′
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1407	[illegible]	[illegible]	4
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	6
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	7
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	8
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	9
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Table of Chords (Continued). Radius = 1.0000

M.	11°	12°	13°	14°	15°	16°	17°	18°	19°	20°	21°
0′	0.1917	0.2091	0.2264	0.2437	0.2611	0.2783	0.2956	0.3129	0.3301	0.3473	0.364
1	0.1920	0.2093	0.2267	0.2440	0.2613	0.2786	0.2959	0.3132	0.3304	0.3476	0.364
2	0.1923	0.2096	0.2270	0.2443	0.2616	0.2789	0.2962	0.3134	0.3307	0.3479	0.365
3	0.1926	0.2099	0.2273	0.2446	0.2619	0.2792	0.2965	0.3137	0.3310	0.3482	0.365
4	0.1928	0.2102	0.2276	0.2449	0.2622	0.2795	0.2968	0.3140	0.3312	0.3484	0.365
5	0.1931	0.2105	0.2279	0.2452	0.2625	0.2798	0.2971	0.3143	0.3315	0.3487	0.365
6	0.1934	0.2108	0.2281	0.2455	0.2628	0.2801	0.2973	0.3146	0.3318	0.3490	0.366
7	0.1937	0.2111	0.2284	0.2458	0.2631	0.2804	0.2976	0.3149	0.3321	0.3493	0.366
8	0.1940	0.2114	0.2287	0.2460	0.2634	0.2807	0.2979	0.3152	0.3324	0.3496	0.366
9	0.1943	0.2117	0.2290	0.2463	0.2636	0.2809	0.2982	0.3155	0.3327	0.3499	0.367
10	0.1946	0.2119	0.2293	0.2466	0.2639	0.2812	0.2985	0.3157	0.3330	0.3502	0.367
11	0.1949	0.2122	0.2296	0.2469	0.2642	0.2815	0.2988	0.3160	0.3333	0.3504	0.367
12	0.1952	0.2125	0.2299	0.2472	0.2645	0.2818	0.2991	0.3163	0.3335	0.3507	0.367
13	0.1955	0.2128	0.2302	0.2475	0.2648	0.2821	0.2994	0.3166	0.3338	0.3510	0.368
14	0.1957	0.2131	0.2305	0.2478	0.2651	0.2824	0.2996	0.3169	0.3341	0.3513	0.368
15	0.1960	0.2134	0.2307	0.2481	0.2654	0.2827	0.2999	0.3172	0.3344	0.3516	0.368
16	0.1963	0.2137	0.2310	0.2484	0.2657	0.2830	0.3002	0.3175	0.3347	0.3519	0.369
17	0.1966	0.2140	0.2313	0.2486	0.2660	0.2832	0.3005	0.3178	0.3350	0.3522	0.369
18	0.1969	0.2143	0.2316	0.2489	0.2662	0.2835	0.3008	0.3180	0.3353	0.3525	0.369
19	0.1972	0.2146	0.2319	0.2492	0.2665	0.2838	0.3011	0.3183	0.3355	0.3527	0.369
20	0.1975	0.2148	0.2322	0.2495	0.2668	0.2841	0.3014	0.3186	0.3358	0.3530	0.370
21	0.1978	0.2151	0.2325	0.2498	0.2671	0.2844	0.3017	0.3189	0.3361	0.3533	0.370
22	0.1981	0.2154	0.2328	0.2501	0.2674	0.2847	0.3019	0.3192	0.3364	0.3536	0.370
23	0.1983	0.2157	0.2331	0.2504	0.2677	0.2850	0.3022	0.3195	0.3367	0.3539	0.371
24	0.1986	0.2160	0.2333	0.2507	0.2680	0.2853	0.3025	0.3198	0.3370	0.3542	0.371
25	0.1989	0.2163	0.2336	0.2510	0.2683	0.2855	0.3028	0.3200	0.3373	0.3545	0.371
26	0.1992	0.2166	0.2339	0.2512	0.2685	0.2858	0.3031	0.3203	0.3376	0.3547	0.371
27	0.1995	0.2169	0.2342	0.2515	0.2688	0.2861	0.3034	0.3206	0.3378	0.3550	0.372
28	0.1998	0.2172	0.2345	0.2518	0.2691	0.2864	0.3037	0.3209	0.3381	0.3553	0.372
29	0.2001	0.2174	0.2348	0.2521	0.2694	0.2867	0.3040	0.3212	0.3384	0.3556	0.372
30	0.2004	0.2177	0.2351	0.2524	0.2697	0.2870	0.3042	0.3215	0.3387	0.3559	0.373
31	0.2007	0.2180	0.2354	0.2527	0.2700	0.2873	0.3045	0.3218	0.3390	0.3562	0.373
32	0.2010	0.2183	0.2357	0.2530	0.2703	0.2876	0.3048	0.3221	0.3393	0.3565	0.373
33	0.2012	0.2186	0.2359	0.2533	0.2706	0.2878	0.3051	0.3223	0.3396	0.3567	0.373
34	0.2015	0.2189	0.2362	0.2536	0.2709	0.2881	0.3054	0.3226	0.3398	0.3570	0.374
35	0.2018	0.2192	0.2365	0.2538	0.2711	0.2884	0.3057	0.3229	0.3401	0.3573	0.374
36	0.2021	0.2195	0.2368	0.2541	0.2714	0.2887	0.3060	0.3232	0.3404	0.3576	0.374
37	0.2024	0.2198	0.2371	0.2544	0.2717	0.2890	0.3063	0.3235	0.3407	0.3579	0.375
38	0.2027	0.2200	0.2374	0.2547	0.2720	0.2893	0.3065	0.3238	0.3410	0.3582	0.375
39	0.2030	0.2203	0.2377	0.2550	0.2723	0.2896	0.3068	0.3241	0.3413	0.3585	0.375
40	0.2033	0.2206	0.2380	0.2553	0.2726	0.2899	0.3071	0.3244	0.3416	0.3587	0.375
41	0.2036	0.2209	0.2383	0.2556	0.2729	0.2902	0.3074	0.3246	0.3419	0.3590	0.376
42	0.2038	0.2212	0.2385	0.2559	0.2732	0.2904	0.3077	0.3249	0.3421	0.3593	0.376
43	0.2041	0.2215	0.2388	0.2561	0.2734	0.2907	0.3080	0.3252	0.3424	0.3596	0.376
44	0.2044	0.2218	0.2391	0.2564	0.2737	0.2910	0.3083	0.3255	0.3427	0.3599	0.377
45	0.2047	0.2221	0.2394	0.2567	0.2740	0.2913	0.3086	0.3258	0.3430	0.3602	0.377
46	0.2050	0.2224	0.2397	0.2570	0.2743	0.2916	0.3088	0.3261	0.3433	0.3605	0.377
47	0.2053	0.2226	0.2400	0.2573	0.2746	0.2919	0.3091	0.3264	0.3436	0.3608	0.377
48	0.2056	0.2229	0.2403	0.2576	0.2749	0.2922	0.3094	0.3267	0.3439	0.3610	0.378
49	0.2059	0.2232	0.2406	0.2579	0.2752	0.2925	0.3097	0.3269	0.3441	0.3613	0.378
50	0.2062	0.2235	0.2409	0.2582	0.2755	0.2927	0.3100	0.3272	0.3444	0.3616	0.378
51	0.2065	0.2238	0.2411	0.2585	0.2758	0.2930	0.3103	0.3275	0.3447	0.3619	0.379
52	0.2067	0.2241	0.2414	0.2587	0.2760	0.2933	0.3106	0.3278	0.3450	0.3522	0.379
53	0.2070	0.2244	0.2417	0.2590	0.2763	0.2936	0.3109	0.3281	0.3453	0.3625	0.379
54	0.2073	0.2247	0.2420	0.2593	0.2766	0.2939	0.3111	0.3284	0.3456	0.3628	0.379
55	0.2076	0.2250	0.2423	0.2596	0.2769	0.2942	0.3114	0.3287	0.3459	0.3630	0.380
56	0.2079	0.2253	0.2426	0.2599	0.2772	0.2945	0.3117	0.3289	0.3462	0.3633	0.380
57	0.2082	0.2255	0.2429	0.2602	0.2775	0.2948	0.3120	0.3292	0.3464	0.3636	0.380
58	0.2085	0.2258	0.2432	0.2605	0.2778	0.2950	0.3123	0.3295	0.3467	0.3639	0.381
59	0.2088	0.2261	0.2434	0.2608	0.2781	0.2953	0.3126	0.3298	0.3470	0.3642	0.381
60	0.2091	0.2264	0.2437	0.2611	0.2783	0.2956	0.3129	0.3301	0.3473	0.3645	0.381

Table of Chords

Table of Chords (Continued). Radius = 1.0000

22°	23°	24°	25°	26°	27°	28°	29°	30°	31°	32°
0.3816	0.3987	0.4158	0.4329	0.4499	0.4669	0.4838	0.5008	0.5176	0.5345	0.5513
0.3819	0.3990	0.4161	0.4332	0.4502	0.4672	0.4841	0.5010	0.5179	0.5348	0.5516
0.3822	0.3993	0.4164	0.4334	0.4505	0.4675	0.4844	0.5013	0.5182	0.5350	0.5518
0.3825	0.3996	0.4167	0.4337	0.4508	0.4677	0.4847	0.5016	0.5185	0.5353	0.5521
0.3828	0.3999	0.4170	0.4340	0.4510	0.4680	0.4850	0.5019	0.5188	0.5356	0.5524
0.3830	0.4002	0.4172	0.4343	0.4513	0.4683	0.4853	0.5022	0.5190	0.5359	0.5527
0.3833	0.4004	0.4175	0.4346	0.4516	0.4686	0.4855	0.5024	0.5193	0.5362	0.5530
0.3836	0.4007	0.4178	0.4349	0.4519	0.4689	0.4858	0.5027	0.5196	0.5364	0.5532
0.3839	0.4010	0.4181	0.4352	0.4522	0.4692	0.4861	0.5030	0.5199	0.5367	0.5535
0.3842	0.4013	0.4184	0.4354	0.4525	0.4694	0.4864	0.5033	0.5202	0.5370	0.5538
0.3845	0.4016	0.4187	0.4357	0.4527	0.4697	0.4867	0.5036	0.5204	0.5373	0.5541
0.3848	0.4019	0.4190	0.4360	0.4530	0.4700	0.4869	0.5039	0.5207	0.5376	0.5543
0.3850	0.4022	0.4192	0.4363	0.4533	0.4703	0.4872	0.5041	0.5210	0.5378	0.5546
0.3853	0.4024	0.4195	0.4366	0.4536	0.4706	0.4875	0.5044	0.5213	0.5381	0.5549
0.3856	0.4027	0.4198	0.4369	0.4539	0.4708	0.4878	0.5047	0.5216	0.5384	0.5552
0.3859	0.4030	0.4201	0.4371	0.4542	0.4711	0.4881	0.5050	0.5219	0.5387	0.5555
0.3862	0.4033	0.4204	0.4374	0.4544	0.4714	0.4884	0.5053	0.5221	0.5390	0.5557
0.3865	0.4036	0.4207	0.4377	0.4547	0.4717	0.4886	0.5055	0.5224	0.5392	0.5560
0.3868	0.4039	0.4209	0.4380	0.4550	0.4720	0.4889	0.5058	0.5227	0.5395	0.5563
0.3870	0.4042	0.4212	0.4383	0.4553	0.4723	0.4892	0.5061	0.5230	0.5398	0.5566
0.3873	0.4044	0.4215	0.4386	0.4556	0.4725	0.4895	0.5064	0.5233	0.5401	0.5569
0.3876	0.4047	0.4218	0.4388	0.4559	0.4728	0.4898	0.5067	0.5235	0.5404	0.5571
0.3879	0.4050	0.4221	0.4391	0.4561	0.4731	0.4901	0.5070	0.5238	0.5406	0.5574
0.3882	0.4053	0.4224	0.4394	0.4564	0.4734	0.4903	0.5072	0.5241	0.5409	0.5577
0.3885	0.4056	0.4226	0.4397	0.4567	0.4737	0.4906	0.5075	0.5244	0.5412	0.5580
0.3888	0.4059	0.4229	0.4400	0.4570	0.4740	0.4909	0.5078	0.5247	0.5415	0.5583
0.3890	0.4061	0.4232	0.4403	0.4573	0.4742	0.4912	0.5081	0.5249	0.5418	0.5585
0.3893	0.4064	0.4235	0.4405	0.4576	0.4745	0.4915	0.5084	0.5252	0.5420	0.5588
0.3896	0.4067	0.4238	0.4408	0.4578	0.4748	0.4917	0.5086	0.5255	0.5423	0.5591
0.3899	0.4070	0.4241	0.4411	0.4581	0.4751	0.4920	0.5089	0.5258	0.5426	0.5594
0.3902	0.4073	0.4244	0.4414	0.4584	0.4754	0.4923	0.5092	0.5261	0.5429	0.5597
0.3905	0.4076	0.4246	0.4417	0.4587	0.4757	0.4926	0.5095	0.5263	0.5432	0.5599
0.3908	0.4079	0.4249	0.4420	0.4590	0.4759	0.4929	0.5098	0.5266	0.5434	0.5602
0.3910	0.4081	0.4252	0.4422	0.4593	0.4762	0.4932	0.5100	0.5269	0.5437	0.5605
0.3913	0.4084	0.4255	0.4425	0.4595	0.4765	0.4934	0.5103	0.5272	0.5440	0.5608
0.3916	0.4087	0.4258	0.4428	0.4598	0.4768	0.4937	0.5106	0.5275	0.5443	0.5611
0.3919	0.4090	0.4261	0.4431	0.4601	0.4771	0.4940	0.5109	0.5277	0.5446	0.5613
0.3922	0.4093	0.4263	0.4434	0.4604	0.4773	0.4943	0.5112	0.5280	0.5448	0.5616
0.3925	0.4096	0.4266	0.4437	0.4607	0.4776	0.4946	0.5115	0.5283	0.5451	0.5619
0.3927	0.4098	0.4269	0.4439	0.4609	0.4779	0.4948	0.5117	0.5286	0.5454	0.5622
0.3930	0.4101	0.4272	0.4442	0.4612	0.4782	0.4951	0.5120	0.5289	0.5457	0.5625
0.3933	0.4104	0.4275	0.4445	0.4615	0.4785	0.4954	0.5123	0.5291	0.5460	0.5627
0.3936	0.4107	0.4278	0.4448	0.4618	0.4788	0.4957	0.5126	0.5294	0.5462	0.5630
0.3939	0.4110	0.4280	0.4451	0.4621	0.4790	0.4960	0.5129	0.5297	0.5465	0.5633
0.3942	0.4113	0.4283	0.4454	0.4624	0.4793	0.4963	0.5131	0.5300	0.5468	0.5636
0.3945	0.4116	0.4286	0.4456	0.4626	0.4796	0.4965	0.5134	0.5303	0.5471	0.5638
0.3947	0.4118	0.4289	0.4459	0.4629	0.4799	0.4968	0.5137	0.5306	0.5474	0.5641
0.3950	0.4121	0.4292	0.4462	0.4632	0.4802	0.4971	0.5140	0.5308	0.4576	0.5644
0.3953	0.4124	0.4295	0.4465	0.4635	0.4805	0.4974	0.5143	0.5311	0.5479	0.5647
0.3956	0.4127	0.4298	0.4468	0.4638	0.4807	0.4977	0.5145	0.5314	0.5482	0.5650
0.3959	0.4130	0.4300	0.4471	0.4641	0.4810	0.4979	0.5148	0.5317	0.5485	0.5652
0.3962	0.4133	0.4303	0.4474	0.4643	0.4813	0.4982	0.5151	0.5320	0.5488	0.5655
0.3965	0.4135	0.4306	0.4476	0.4646	0.4816	0.4985	0.5154	0.5322	0.5490	0.5658
0.3967	0.4138	0.4309	0.4479	0.4649	0.4819	0.4988	0.5157	0.5325	0.5493	0.5661
0.3970	0.4141	0.4312	0.4482	0.4652	0.4822	0.4991	0.5160	0.5328	0.5496	0.5664
0.3973	0.4144	0.4315	0.4485	0.4655	0.4824	0.4994	0.5162	0.5331	0.5499	0.5666
0.3976	0.4147	0.4317	0.4488	0.4658	0.4827	0.4996	0.5165	0.5334	0.5502	0.5669
0.3979	0.4150	0.4320	0.4491	0.4660	0.4830	0.4999	0.5168	0.5336	0.5504	0.5672
0.3982	0.4153	0.4323	0.4493	0.4663	0.4833	0.5002	0.5171	0.5339	0.5507	0.5675
0.3985	0.4155	0.4326	0.4496	0.4666	0.4836	0.5005	0.5174	0.5342	0.5510	0.5678
0.3987	0.4158	0.4329	0.4499	0.4669	0.4838	0.5008	0.5176	0.5345	0.5513	0.5680

Table of Chords (Continued). Radius = 1.0000

33°	34°	35°	36°	37°	38°	39°	40°	41°	42°	43°
0.5680	0.5847	0.6014	0.6180	0.6346	0.6511	0.6676	0.6840	0.7004	0.7167	0.7330
0.5683	0.5850	0.6017	0.6183	0.6349	0.6514	0.6679	0.6843	0.7007	0.7170	0.7333
0.5686	0.5853	0.6020	0.6186	0.6352	0.6517	0.6682	0.6846	0.7010	0.7173	0.7335
0.5689	0.5856	0.6022	0.6189	0.6354	0.6520	0.6684	0.6849	0.7012	0.7176	0.7338
0.5691	0.5859	0.6025	0.6191	0.6357	0.6522	0.6687	0.6851	0.7015	0.7178	0.7341
0.5694	0.5861	0.6028	0.6194	0.6360	0.6525	0.6690	0.6854	0.7018	0.7181	0.7344
0.5697	0.5864	0.6031	0.6197	0.6363	0.6528	0.6693	0.6857	0.7020	0.7184	0.7346
0.5700	0.5867	0.6034	0.6200	0.6365	0.6531	0.6695	0.6860	0.7023	0.7186	0.7349
0.5703	0.5870	0.6036	0.6202	0.6368	0.6533	0.6698	0.6862	0.7026	0.7189	0.7352
0.5705	0.5872	0.6039	0.6205	0.6371	0.6536	0.6701	0.6865	0.7029	0.7192	0.7354
0.5708	0.5875	0.6042	0.6208	0.6374	0.6539	0.6704	0.6868	0.7031	0.7195	0.7357
0.5711	0.5878	0.6045	0.6211	0.6376	0.6542	0.6706	0.6870	0.7034	0.7197	0.7360
0.5714	0.5881	0.6047	0.6214	0.6379	0.6544	0.6709	0.6873	0.7037	0.7200	0.7363
0.5717	0.5884	0.6050	0.6216	0.6382	0.6547	0.6712	0.6876	0.7040	0.7203	0.7365
0.5719	0.5886	0.6053	0.6219	0.6385	0.6550	0.6715	0.6879	0.7042	0.7206	0.7368
0.5722	0.5889	0.6056	0.6222	0.6387	0.6553	0.6717	0.6881	0.7045	0.7208	0.7371
0.5725	0.5892	0.6058	0.6225	0.6390	0.6555	0.6720	0.6884	0.7048	0.7211	0.7373
0.5728	0.5895	0.6061	0.6227	0.6393	0.6558	0.6723	0.6887	0.7050	0.7214	0.7376
0.5730	0.5897	0.6064	0.6230	0.6396	0.6561	0.6725	0.6890	0.7053	0.7216	0.7379
0.5733	0.5900	0.6067	0.6233	0.6398	0.6564	0.6728	0.6892	0.7056	0.7219	0.7381
0.5736	0.5903	0.6070	0.6236	0.6401	0.6566	0.6731	0.6895	0.7059	0.7222	0.7384
0.5739	0.5906	0.6072	0.6238	0.6404	0.6569	0.6734	0.6898	0.7061	0.7224	0.7387
0.5742	0.5909	0.6075	0.6241	0.6407	0.6572	0.6736	0.6901	0.7064	0.7227	0.7390
0.5744	0.5911	0.6078	0.6244	0.6410	0.6575	0.6739	0.6903	0.7067	0.7230	0.7392
0.5747	0.5914	0.6081	0.6247	0.6412	0.6577	0.6742	0.6906	0.7069	0.7232	0.7395
0.5750	0.5917	0.6083	0.6249	0.6415	0.6580	0.6745	0.6909	0.7072	0.7235	0.7398
0.5753	0.5920	0.6086	0.6252	0.6418	0.6583	0.6747	0.6911	0.7075	0.7238	0.7400
0.5756	0.5922	0.6089	0.6255	0.6421	0.6586	0.6750	0.6914	0.7078	0.7241	0.7403
0.5758	0.5925	0.6092	0.6258	0.6423	0.6588	0.6753	0.6917	0.7080	0.7243	0.7406
0.5761	0.5928	0.6095	0.6260	0.6426	0.6591	0.6756	0.6920	0.7083	0.7246	0.7408
0.5764	0.5931	0.6097	0.6263	0.6429	0.6594	0.6758	0.6922	0.7086	0.7249	0.7411
0.5767	0.5934	0.6100	0.6266	0.6432	0.6597	0.6761	0.6925	0.7089	0.7251	0.7414
0.5769	0.5936	0.6103	0.6269	0.6434	0.6599	0.6764	0.6928	0.7091	0.7254	0.7417
0.5772	0.5939	0.6106	0.6272	0.6437	0.6602	0.6767	0.6931	0.7094	0.7257	0.7419
0.5775	0.5942	0.6108	0.6274	0.6440	0.6605	0.6769	0.6933	0.7097	0.7260	0.7422
0.5778	0.5945	0.6111	0.6277	0.6443	0.6608	0.6772	0.6936	0.7099	0.7262	0.7425
0.5781	0.5947	0.6114	0.6280	0.6445	0.6610	0.6775	0.6939	0.7102	0.7265	0.7427
0.5783	0.5950	0.6117	0.6283	0.6448	0.6613	0.6777	0.6941	0.7105	0.7268	0.7430
0.5786	0.5953	0.6119	0.6285	0.6451	0.6616	0.6780	0.6944	0.7108	0.7270	0.7433
0.5789	0.5956	0.6122	0.6288	0.6454	0.6619	0.6783	0.6947	0.7110	0.7273	0.7435
0.5792	0.5959	0.6125	0.6291	0.6456	0.6621	0.6786	0.6950	0.7113	0.7276	0.7438
0.5795	0.5961	0.6128	0.6294	0.6459	0.6624	0.6788	0.6952	0.7116	0.7279	0.7441
0.5797	0.5964	0.6130	0.6296	0.6462	0.6627	0.6791	0.6955	0.7118	0.7281	0.7443
0.5800	0.5967	0.6133	0.6299	0.6465	0.6630	0.6794	0.6958	0.7121	0.7284	0.7446
0.5803	0.5970	0.6136	0.6302	0.6467	0.6632	0.6797	0.6961	0.7124	0.7287	0.7449
0.5806	0.5972	0.6139	0.6305	0.6470	0.6635	0.6799	0.6963	0.7127	0.7289	0.7452
0.5808	0.5975	0.6142	0.6307	0.6473	0.6638	0.6802	0.6966	0.7129	0.7292	0.7454
0.5811	0.5978	0.6144	0.6310	0.6476	0.6640	0.6805	0.6969	0.7132	0.7295	0.7457
0.5814	0.5981	0.6147	0.6313	0.6478	0.6643	0.6808	0.6971	0.7135	0.7298	0.7460
0.5817	0.5984	0.6150	0.6316	0.6481	0.6646	0.6810	0.6974	0.7137	0.7300	0.7462
0.5820	0.5986	0.6153	0.6318	0.6484	0.6649	0.6813	0.6977	0.7140	0.7303	0.7465
0.5822	0.5989	0.6155	0.6321	0.6487	0.6651	0.6816	0.6980	0.7143	0.7306	0.7468
0.5825	0.5992	0.6158	0.6324	0.6489	0.6654	0.6819	0.6982	0.7146	0.7308	0.7471
0.5828	0.5995	0.6161	0.6327	0.6492	0.6657	0.6821	0.6985	0.7148	0.7311	0.7473
0.5831	0.5997	0.6164	0.6330	0.6495	0.6660	0.6824	0.6988	0.7151	0.7314	0.7476
0.5834	0.6000	0.6166	0.6332	0.6498	0.6662	0.6827	0.6991	0.7154	0.7316	0.7479
0.5836	0.6003	0.6169	0.6335	0.6500	0.6665	0.6829	0.6993	0.7156	0.7319	0.7481
0.5839	0.6006	0.6172	0.6338	0.6503	0.6668	0.6832	0.6996	0.7159	0.7322	0.7484
0.5842	0.6009	0.6175	0.6341	0.6506	0.6671	0.6835	0.6999	0.7162	0.7325	0.7487
0.5845	0.6011	0.6178	0.6343	0.6509	0.6673	0.6838	0.7001	0.7165	0.7327	0.7489
0.5847	0.6014	0.6180	0.6346	0.6511	0.6676	0.6840	0.7004	0.7167	0.7330	0.7492

Table of Chords (Continued). Radius = 1.0000

M'	45°	46°	47°	48°	49°	50°	51°	52°	53°	54°
[illegible]	0.7654	0.7815	0.7975	0.8135	0.8294	0.8452	0.8610	0.8767	0.8924	0.9080
[illegible]	0.7656	0.7817	0.7978	0.8137	0.8297	0.8455	0.8613	0.8770	0.8927	0.9082
[illegible]	0.7659	0.7820	0.7980	0.8140	0.8299	0.8458	0.8615	0.8773	0.8929	0.9085
[illegible]	0.7662	0.7823	0.7983	0.8143	0.8302	0.8460	0.8618	0.8775	0.8932	0.9088
[illegible]	0.7664	0.7825	0.7986	0.8145	0.8304	0.8463	0.8621	0.8778	0.8934	0.9090
[illegible]	0.7667	0.7828	0.7988	0.8148	0.8307	0.8466	0.8623	0.8780	0.8937	0.9093
[illegible]	0.7670	0.7831	0.7991	0.8151	0.8310	0.8468	0.8626	0.8783	0.8940	0.9095
[illegible]	0.7672	0.7833	0.7994	0.8153	0.8312	0.8471	0.8629	0.8786	0.8942	0.9098
[illegible]	0.7675	0.7836	0.7996	0.8156	0.8315	0.8473	0.8631	0.8788	0.8945	0.9101
[illegible]	0.7678	0.7839	0.7999	0.8159	0.8318	0.8476	0.8634	0.8791	0.8947	0.9103
[illegible]	0.7681	0.7841	0.8002	0.8161	0.8320	0.8479	0.8636	0.8794	0.8950	0.9106
[illegible]	0.7683	0.7844	0.8004	0.8164	0.8323	0.8481	0.8639	0.8796	0.8953	0.9108
[illegible]	0.7686	0.7847	0.8007	0.8167	0.8326	0.8484	0.8642	0.8799	0.8955	0.9111
[illegible]	0.7689	0.7849	0.8010	0.8169	0.8328	0.8487	0.8644	0.8801	0.8958	0.9113
[illegible]	0.7691	0.7852	0.8012	0.8172	0.8331	0.8489	0.8647	0.8804	0.8960	0.9116
[illegible]	0.7694	0.7855	0.8015	0.8175	0.8334	0.8492	0.8650	0.8807	0.8963	0.9119
[illegible]	0.7697	0.7857	0.8018	0.8177	0.8336	0.8495	0.8652	0.8809	0.8966	0.9121
[illegible]	0.7699	0.7860	0.8020	0.8180	0.8339	0.8497	0.8655	0.8812	0.8968	0.9124
[illegible]	0.7702	0.7863	0.8023	0.8183	0.8341	0.8500	0.8657	0.8814	0.8971	0.9126
[illegible]	0.7705	0.7865	0.8026	0.8185	0.8344	0.8502	0.8660	0.8817	0.8973	0.9129
[illegible]	0.7707	0.7868	0.8028	0.8188	0.8347	0.8505	0.8663	0.8820	0.8976	0.9132
[illegible]	0.7710	0.7871	0.8031	0.8190	0.8349	0.8508	0.8665	0.8822	0.8979	0.9134
[illegible]	0.7713	0.7873	0.8034	0.8193	0.8352	0.8510	0.8668	0.8825	0.8981	0.9137
[illegible]	0.7715	0.7876	0.8036	0.8196	0.8355	0.8513	0.8671	0.8828	0.8984	0.9139
[illegible]	0.7718	0.7879	0.8039	0.8198	0.8357	0.8516	0.8673	0.8830	0.8986	0.9142
[illegible]	0.7721	0.7882	0.8042	0.8201	0.8360	0.8518	0.8676	0.8833	0.8989	0.9145
[illegible]	0.7723	0.7884	0.8044	0.8204	0.8363	0.8521	0.8678	0.8835	0.8992	0.9147
[illegible]	0.7726	0.7887	0.8047	0.8206	0.8365	0.8523	0.8681	0.8838	0.8994	0.9150
[illegible]	0.7729	0.7890	0.8050	0.8209	0.8368	0.8526	0.8684	0.8841	0.8997	0.9152
[illegible]	0.7731	0.7892	0.8052	0.8212	0.8371	0.8529	0.8686	0.8843	0.8999	0.9155
[illegible]	0.7734	0.7895	0.8055	0.8214	0.8373	0.8531	0.8689	0.8846	0.9002	0.9157
[illegible]	0.7737	0.7898	0.8058	0.8217	0.8376	0.8534	0.8692	0.8848	0.9005	0.9160
[illegible]	0.7740	0.7900	0.8060	0.8220	0.8378	0.8537	0.8694	0.8851	0.9007	0.9163
[illegible]	0.7742	0.7903	0.8063	0.8222	0.8381	0.8539	0.8697	0.8854	0.9010	0.9165
[illegible]	0.7745	0.7906	0.8066	0.8225	0.8384	0.8542	0.8699	0.8856	0.9012	0.9168
[illegible]	0.7748	0.7908	0.8068	0.8228	0.8386	0.8545	0.8702	0.8859	0.9015	0.9170
[illegible]	0.7750	0.7911	0.8071	0.8230	0.8389	0.8547	0.8705	0.8861	0.9018	0.9173
[illegible]	0.7753	0.7914	0.8074	0.8233	0.8392	0.8550	0.8707	0.8864	0.9020	0.9176
[illegible]	0.7756	0.7916	0.8076	0.8236	0.8394	0.8552	0.8710	0.8867	0.9023	0.9178
[illegible]	0.7758	0.7919	0.8079	0.8238	0.8397	0.8555	0.8712	0.8869	0.9025	0.9181
[illegible]	0.7761	0.7922	0.8082	0.8241	0.8400	0.8558	0.8715	0.8872	0.9028	0.9183
[illegible]	0.7764	0.7924	0.8084	0.8244	0.8402	0.8560	0.8718	0.8874	0.9031	0.9186
[illegible]	0.7766	0.7927	0.8087	0.8246	0.8405	0.8563	0.8720	0.8877	0.9033	0.9188
[illegible]	0.7769	0.7930	0.8090	0.8249	0.8408	0.8566	0.8723	0.8880	0.9036	0.9191
[illegible]	0.7772	0.7932	0.8092	0.8251	0.8410	0.8568	0.8726	0.8882	0.9038	0.9194
[illegible]	0.7774	0.7935	0.8095	0.8254	0.8413	0.8571	0.8728	0.8885	0.9041	0.9196
[illegible]	0.7777	0.7938	0.8098	0.8257	0.8415	0.8573	0.8731	0.8887	0.9044	0.9199
[illegible]	0.7780	0.7940	0.8100	0.8259	0.8418	0.8576	0.8734	0.8890	0.9046	0.9201
[illegible]	0.7782	0.7943	0.8103	0.8262	0.8421	0.8579	0.8736	0.8893	0.9049	0.9204
[illegible]	0.7785	0.7946	0.8105	0.8265	0.8423	0.8581	0.8739	0.8895	0.9051	0.9207
[illegible]	0.7788	0.7948	0.8108	0.8267	0.8426	0.8584	0.8741	0.8898	0.9054	0.9209
[illegible]	0.7791	0.7951	0.8111	0.8270	0.8429	0.8587	0.8744	0.8900	0.9056	0.9212
[illegible]	0.7793	0.7954	0.8113	0.8273	0.8431	0.8589	0.8747	0.8903	0.9059	0.9214
[illegible]	0.7796	0.7956	0.8116	0.8275	0.8434	0.8592	0.8749	0.8906	0.9062	0.9217
[illegible]	0.7799	0.7959	0.8119	0.8278	0.8437	0.8594	0.8752	0.8908	0.9064	0.9219
[illegible]	0.7801	0.7962	0.8121	0.8281	0.8439	0.8597	0.8754	0.8911	0.9067	0.9222
[illegible]	0.7804	0.7964	0.8124	0.8283	0.8442	0.8600	0.8757	0.8914	0.9069	0.9225
[illegible]	0.7807	0.7967	0.8127	0.8286	0.8444	0.8602	0.8760	0.8916	0.9072	0.9227
[illegible]	0.7809	0.7970	0.8129	0.8289	0.8447	0.8605	0.8762	0.8919	0.9075	0.9230
[illegible]	0.7812	0.7972	0.8132	0.8291	0.8450	0.8608	0.8765	0.8921	0.9077	0.9232
[illegible]	0.7815	0.7975	0.8135	0.8294	0.8452	0.8610	0.8767	0.8924	0.9080	0.9235

Table of Chords (Continued). Radius = 1.0000

M.	55°	56°	57°	58°	59°	60°	61°	62°	63°	(
0′	0.9235	0.9389	0.9543	0.9696	0.9848	1.0000	1.0151	1.0301	1.0450	1.
1	0.9238	0.9392	0.9546	0.9699	0.9851	1.0003	1.0153	1.0303	1.0452	1.
2	0.9240	0.9395	0.9548	0.9701	0.9854	1.0005	1.0156	1.0306	1.0455	1.
3	0.9243	0.9397	0.9551	0.9704	0.9856	1.0008	1.0158	1.0308	1.0457	1.
4	0.9245	0.9400	0.9553	0.9706	0.9859	1.0010	1.0161	1.0311	1.0460	1.
5	0.9248	0.9402	0.9556	0.9709	0.9861	1.0013	1.0163	1.0313	1.0462	1.
6	0.9250	0.9405	0.9559	0.9711	0.9864	1.0015	1.0166	1.0316	1.0465	1.
7	0.9253	0.9407	0.9561	0.9714	0.9866	1.0018	1.0168	1.0318	1.0467	1.
8	0.9256	0.9410	0.9564	0.9717	0.9869	1.0020	1.0171	1.0321	1.0470	1.
9	0.9258	0.9413	0.9566	0.9719	0.9871	1.0023	1.0173	1.0323	1.0472	1.
10	0.9261	0.9415	0.9569	0.9722	0.9874	1.0025	1.0176	1.0326	1.0475	1.
11	0.9263	0.9418	0.9571	0.9724	0.9876	1.0028	1.0178	1.0328	1.0477	1.
12	0.9266	0.9420	0.9574	0.9727	0.9879	1.0030	1.0181	1.0331	1.0480	1.
13	0.9268	0.9423	0.9576	0.9729	0.9881	1.0033	1.0183	1.0333	1.0482	1.
14	0.9271	0.9425	0.9579	0.9732	0.9884	1.0035	1.0186	1.0336	1.0485	1.
15	0.9274	0.9428	0.9581	0.9734	0.9886	1.0038	1.0188	1.0338	1.0487	1.
16	0.9276	0.9430	0.9584	0.9737	0.9889	1.0040	1.0191	1.0341	1.0490	1.
17	0.9279	0.9433	0.9587	0.9739	0.9891	1.0043	1.0193	1.0343	1.0492	1.
18	0.9281	0.9436	0.9589	0.9742	0.9894	1.0045	1.0196	1.0346	1.0495	1.
19	0.9284	0.9438	0.9592	0.9744	0.9897	1.0048	1.0198	1.0348	1.0497	1.
20	0.9287	0.9441	0.9594	0.9747	0.9899	1.0050	1.0201	1.0351	1.0500	1.
21	0.9289	0.9443	0.9597	0.9750	0.9902	1.0053	1.0203	1.0353	1.0502	1.
22	0.9292	0.9446	0.9599	0.9752	0.9904	1.0055	1.0206	1.0356	1.0504	1.
23	0.9294	0.9448	0.9602	0.9755	0.9907	1.0058	1.0208	1.0358	1.0507	1.
24	0.9297	0.9451	0.9604	0.9757	0.9909	1.0060	1.0211	1.0361	1.0509	1.
25	0.9299	0.9454	0.9607	0.9760	0.9912	1.0063	1.0213	1.0363	1.0512	1.
26	0.9302	0.9456	0.9610	0.9762	0.9914	1.0065	1.0216	1.0366	1.0514	1.
27	0.9305	0.9459	0.9612	0.9765	0.9917	1.0068	1.0218	1.0368	1.0517	1.
28	0.9307	0.9461	0.9615	0.9767	0.9919	1.0070	1.0221	1.0370	1.0519	1.
29	0.9310	0.9464	0.9617	0.9770	0.9922	1.0073	1.0223	1.0373	1.0522	1.
30	0.9312	0.9466	0.9620	0.9772	0.9924	1.0075	1.0226	1.0375	1.0524	1.
31	0.9315	0.9469	0.9622	0.9775	0.9927	1.0078	1.0228	1.0378	1.0527	1.
32	0.9317	0.9472	0.9625	0.9778	0.9929	1.0080	1.0231	1.0380	1.0529	1.
33	0.9320	0.9474	0.9627	0.9780	0.9932	1.0083	1.0233	1.0383	1.0532	1.
34	0.9323	0.9477	0.9630	9.9783	0.9934	1.0086	1.0236	1.0385	1.0534	1.
35	0.9325	0.9479	0.9633	0.9785	0.9937	1.0088	1.0238	1.0388	1.0537	1.
36	0.9328	0.9482	0.9635	0.9788	0.9939	1.0091	1.0241	1.0390	1.0539	1.
37	0.9330	0.9484	0.9638	0.9790	0.9942	1.0093	1.0243	1.0393	1.0542	1.
38	0.9333	0.9487	0.9640	0.9793	0.9945	1.0096	1.0246	1.0395	1.0544	1.
39	0.9335	0.9489	0.9643	0.9795	0.9947	1.0098	1.0248	1.0398	1.0547	1.
40	0.9338	0.9492	0.9645	0.9798	0.9950	1.0101	1.0251	1.0400	1.0549	1.
41	0.9341	0.9495	0.9648	0.9800	0.9952	1.0103	1.0253	1.0403	1.0551	1.
42	0.9343	0.9497	0.9650	0.9803	0.9955	1.0106	1.0256	1.0405	1.0554	1.
43	0.9346	0.9500	0.9653	0.9805	0.9957	1.0108	1.0258	1.0408	1.0556	1.
44	0.9348	0.9502	0.9655	0.9808	0.9960	1.0111	1.0261	1.0410	1.0559	1.
45	0.9351	0.9505	0.9658	0.9810	0.9962	1.0113	1.0263	1.0413	1.0561	1.
46	0.9353	0.9507	0.9661	0.9813	0.9965	1.0116	1.0266	1.0415	1.0564	1.
47	0.9356	0.9510	0.9663	0.9816	0.9967	1.0118	1.0268	1.0418	1.0566	1.
48	0.9359	0.9512	0.9666	0.9818	0.9970	1.0121	1.0271	1.0420	1.0569	1.
49	0.9361	0.9515	0.9668	0.9821	0.9972	1.0123	1.0273	1.0423	1.0571	1.
50	0.9364	0.9518	0.9671	0.9823	0.9975	1.0126	1.0276	1.0425	1.0574	1.
51	0.9366	0.9520	0.9673	0.9826	0.9977	1.0128	1.0278	1.0428	1.0576	1.
52	0.9369	0.9523	0.9676	0.9828	0.9980	1.0131	1.0281	1.0430	1.0579	1.
53	0.9371	0.9525	0.9678	0.9831	0.9982	1.0133	1.0283	1.0433	1.0581	1.
54	0.9374	0.9528	0.9681	0.9833	0.9985	1.0136	1.0286	1.0435	1.0584	1.
55	0.9377	0.9530	0.9683	0.9836	0.9987	1.0138	1.0288	1.0438	1.0586	1.
56	0.9379	0.9533	0.9686	0.9838	0.9990	1.0141	1.0291	1.0440	1.0589	1.
57	0.9382	0.9536	0.9689	0.9841	0.9992	1.0143	1.0293	1.0443	1.0591	1.
58	0.9384	0.9538	0.9691	0.9843	0.9995	1.0146	1.0296	1.0445	1.0593	1.
59	0.9387	0.9541	0.9694	0.9846	0.9998	1.0148	1.0298	1.0447	1.0596	1.
60	0.9389	0.9543	0.9696	0.9848	1.0000	1.0151	1.0301	1.0450	1.0598	1.

Table of Chords (Continued). Radius = 1.0000

[illegible]	66°	67°	68°	69°	70°	71°	72°	73°	M.
[illegible]	1.0893	1.1039	1.1184	1.1328	1.1472	1.1614	1.1756	1.1896	0′
[illegible]	1.0895	1.1041	1.1186	1.1331	1.1474	1.1616	1.1758	1.1899	1
[illegible]	1.0898	1.1044	1.1189	1.1333	1.1476	1.1619	1.1760	1.1901	2
[illegible]	1.0900	1.1046	1.1191	1.1335	1.1479	1.1621	1.1763	1.1903	3
[illegible]	1.0903	1.1048	1.1194	1.1338	1.1481	1.1624	1.1765	1.1906	4
[illegible]	1.0905	1.1051	1.1196	1.1340	1.1483	1.1626	1.1767	1.1908	5
[illegible]	1.0907	1.1053	1.1198	1.1342	1.1486	1.1628	1.1770	1.1910	6
[illegible]	1.0910	1.1056	1.1201	1.1345	1.1488	1.1631	1.1772	1.1913	7
[illegible]	1.0912	1.1058	1.1203	1.1347	1.1491	1.1633	1.1775	1.1915	8
[illegible]	1.0915	1.1016	1.1206	1.1350	1.1493	1.1635	1.1777	1.1917	9
[illegible]	1.0917	1.1063	1.1208	1.1352	1.1495	1.1638	1.1779	1.1920	10
[illegible]	1.0920	1.1065	1.1210	1.1354	1.1498	1.1640	1.1782	1.1922	11
[illegible]	1.0922	1.1068	1.1213	1.1357	1.1500	1.1642	1.1784	1.1924	12
[illegible]	1.0924	1.1070	1.1215	1.1359	1.1502	1.1645	1.1786	1.1927	13
[illegible]	1.0927	1.1073	1.1218	1.1362	1.1505	1.1647	1.1789	1.1929	14
[illegible]	1.0929	1.1075	1.1220	1.1364	1.1507	1.1650	1.1791	1.1931	15
[illegible]	1.0932	1.1078	1.1222	1.1366	1.1510	1.1652	1.1793	1.1934	16
[illegible]	1.0934	1.1080	1.1225	1.1369	1.1512	1.1654	1.1796	1.1936	17
[illegible]	1.0937	1.1082	1.1227	1.1371	1.1514	1.1657	1.1798	1.1938	18
[illegible]	1.0939	1.1085	1.1230	1.1374	1.1517	1.1659	1.1800	1.1941	19
[illegible]	1.0942	1.1087	1.1232	1.1376	1.1519	1.1661	1.1803	1.1943	20
[illegible]	1.0944	1.1090	1.1234	1.1378	1.1522	1.1664	1.1805	1.1946	21
[illegible]	1.0946	1.1092	1.1237	1.1381	1.1524	1.1666	1.1807	1.1948	22
[illegible]	1.0949	1.1094	1.1239	1.1383	1.1526	1.1668	1.1810	1.1950	23
[illegible]	1.0951	1.1097	1.1242	1.1386	1.1529	1.1671	1.1812	1.1952	24
[illegible]	1.0954	1.1099	1.1244	1.1388	1.1531	1.1673	1.1814	1.1955	25
[illegible]	1.0956	1.1102	1.1246	1.1390	1.1533	1.1676	1.1817	1.1957	26
[illegible]	1.0959	1.1104	1.1249	1.1393	1.1536	1.1678	1.1819	1.1959	27
[illegible]	1.0961	1.1107	1.1251	1.1395	1.1538	1.1680	1.1821	1.1962	28
[illegible]	1.0963	1.1109	1.1254	1.1398	1.1541	1.1683	1.1824	1.1964	29
[illegible]	1.0966	1.1111	1.1256	1.1400	1.1543	1.1685	1.1826	1.1966	30
[illegible]	1.0968	1.1114	1.1258	1.1402	1.1545	1.1687	1.1829	1.1969	31
[illegible]	1.0971	1.1116	1.1261	1.1405	1.1548	1.1690	1.1831	1.1971	32
[illegible]	1.0973	1.1119	1.1263	1.1407	1.1550	1.1692	1.1833	1.1973	33
[illegible]	1.0976	1.1121	1.1266	1.1409	1.1552	1.1694	1.1836	1.1976	34
[illegible]	1.0978	1.1123	1.1268	1.1412	1.1555	1.1697	1.1838	1.1978	35
[illegible]	1.0980	1.1126	1.1271	1.1414	1.1557	1.1699	1.1840	1.1980	36
[illegible]	1.0983	1.1128	1.1273	1.1417	1.1560	1.1702	1.1843	1.1983	37
[illegible]	1.0985	1.1131	1.1275	1.1419	1.1562	1.1704	1.1845	1.1985	38
[illegible]	1.0988	1.1133	1.1278	1.1421	1.1564	1.1706	1.1847	1.1987	39
[illegible]	1.0990	1.1136	1.1280	1.1424	1.1567	1.1709	1.1850	1.1990	40
[illegible]	1.0993	1.1138	1.1283	1.1426	1.1569	1.1711	1.1852	1.1992	41
[illegible]	1.0995	1.1140	1.1285	1.1429	1.1571	1.1713	1.1854	1.1994	42
[illegible]	1.0997	1.1143	1.1287	1.1431	1.1574	1.1716	1.1857	1.1997	43
[illegible]	1.1000	1.1145	1.1290	1.1433	1.1576	1.1718	1.1859	1.1999	44
[illegible]	1.1002	1.1148	1.1292	1.1436	1.1579	1.1720	1.1861	1.2001	45
[illegible]	1.1005	1.1150	1.1295	1.1438	1.1581	1.1723	1.1864	1.2004	46
[illegible]	1.1007	1.1152	1.1297	1.1441	1.1583	1.1725	1.1866	1.2006	47
[illegible]	1.1010	1.1155	1.1299	1.1443	1.1586	1.1727	1.1868	1.2008	48
[illegible]	1.1012	1.1157	1.1302	1.1445	1.1588	1.1730	1.1871	1.2011	49
[illegible]	1.1014	1.1160	1.1304	1.1448	1.1590	1.1732	1.1873	1.2013	50
[illegible]	1.1017	1.1162	1.1307	1.1450	1.1593	1.1735	1.1875	1.2015	51
[illegible]	1.1019	1.1165	1.1309	1.1452	1.1595	1.1737	1.1878	1.2018	52
[illegible]	1.1022	1.1167	1.1311	1.1455	1.1598	1.1739	1.1880	1.2020	53
[illegible]	1.1024	1.1169	1.1314	1.1457	1.1600	1.1742	1.1882	1.2022	54
[illegible]	1.1027	1.1172	1.1316	1.1460	1.1602	1.1744	1.1885	1.2025	55
[illegible]	1.1029	1.1174	1.1319	1.1462	1.1605	1.1746	1.1887	1.2027	56
[illegible]	1.1031	1.1177	1.1321	1.1464	1.1607	1.1749	1.1889	1.2029	57
[illegible]	1.1034	1.1179	1.1323	1.1467	1.1609	1.1751	1.1892	1.2032	58
[illegible]	1.1036	1.1181	1.1326	1.1469	1.1612	1.1753	1.1894	1.2034	59
[illegible]	1.1039	1.1184	1.1328	1.1472	1.1614	1.1756	1.1896	1.2036	60

Table of Chords (Continued). Radius = 1.0000

Table of Chords (Concluded). Radius

84°	85°	86°	87°
1.3383	1.3512	1.3640	1.3767
1.3385	1.3514	1.3642	1.3769
1.3387	1.3516	1.3644	1.3771
1.3389	1.3518	1.3646	1.3773
1.3391	1.3520	1.3648	1.3776
1.3393	1.3523	1.3651	1.3778
1.3396	1.3525	1.3653	1.3780
1.3398	1.3527	1.3655	1.3782
1.3400	1.3529	1.3657	1.3784
1.3402	1.3531	1.3659	1.3786
1.3404	1.3533	1.3661	1.3788
1.3406	1.3535	1.3663	1.3790
1.3409	1.3538	1.3665	1.3792
1.3411	1.3540	1.3668	1.3704
1.3413	1.3542	1.3670	1.3797
1.3415	1.3544	1.3672	1.3799
1.3417	1.3546	1.3674	1.3801
1.3419	1.3548	1.3676	1.3803
1.3421	1.3550	1.3678	1.3805
1.3424	1.3552	1.3680	1.3807
1.3426	1.3555	1.3682	1.3809
1.3428	1.3557	1.3685	1.3811
1.3430	1.3559	1.3687	1.3813
1.3432	1.3561	1.3689	1.3816
1.3434	1.3563	1.3691	1.3818
1.3437	1.3565	1.3693	1.3820
1.3439	1.3567	1.3695	1.3822
1.3441	1.3570	1.3697	1.3824
1.3443	1.3572	1.3699	1.3826
1.3445	1.3574	1.3702	1.3828
1.3447	1.3576	1.3704	1.3830
1.3449	1.3578	1.3706	1.3832
1.3452	1.3580	1.3708	1.3834
1.3454	1.3582	1.3710	1.3837
1.3456	1.3585	1.3712	1.3839
1.3458	1.3587	1.3714	1.3841
1.3460	1.3589	1.3716	1.3843
1.3462	1.3591	1.3718	1.3845
1.3465	1.3593	1.3721	1.3847
1.3467	1.3595	1.3723	1.3849
1.3469	1.3597	1.3725	1.3851
1.3471	1.3599	1.3727	1.3853
1.3473	1.3602	1.3729	1.3855
1.3475	1.3604	1.3731	1.3858
1.3477	1.3606	1.3733	1.3860
1.3480	1.3608	1.3735	1.3862
1.3482	1.3610	1.3738	1.3864
1.3484	1.3612	1.3740	1.3866
1.3486	1.3614	1.3742	1.3868
1.3488	1.3617	1.3744	1.3870
1.3490	1.3619	1.3746	1.3872
1.3492	1.3621	1.3748	1.3874
1.3495	1.3623	1.3750	1.3876
1.3497	1.3625	1.3752	1.3879
1.3499	1.3627	1.3754	1.3881
1.3501	1.3629	1.3757	1.3883
1.3503	1.3631	1.3759	1.3885
1.3505	1.3634	1.3761	1.3887
1.3506	1.3636	1.3763	1.3889
1.3510	1.3638	1.3765	1.3891
1.3512	1.3640	1.3767	1.3893

Lengths and Bevels of Hip-Rafters and Jack-Rafters

Method of Determining the Lengths and Bevels. The lines a (Fig. 92) represent the outside of the walls at the angle of a building; seat of the hip-rafter and *gf* of a jack-rafter. Draw *eh* at right-ang and make it equal to the rise of the roof; join *b* and *h* and *hb* will be t of the hip-rafter. Through *e* draw *di* at right-angles to *bc*. With *b* as

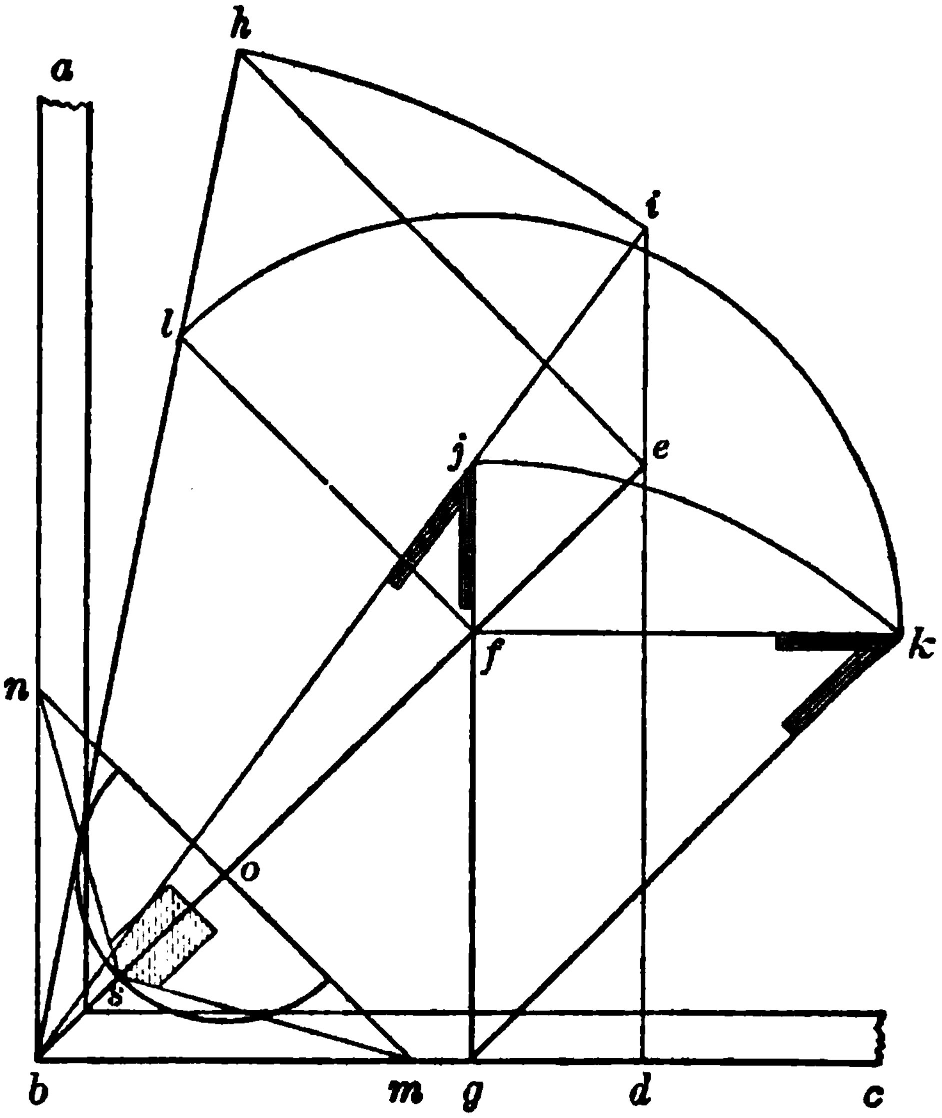

Fig. 92. Lengths and Bevels of Hip-rafters and Jack-rafters

and with the radius *bh*, describe the arc *hi*, cutting *di* in *i*. Join *b* an extend *gf* to meet *bi* in *j*; then *gj* is the length of the jack-rafter. The l each jack-rafter is found in the same manner, by extending its seat to line *bi*. From *f* draw *fk* at right-angles to *fg*; also *fl* at right-angles to *be*. *fk* equal to *fl* by the arc *lk*, or make *gk* equal to *gj* by the arc *jk*; then t at *j* is the TOP BEVEL of the jack-rafters, and the angle at *k* the DOWN BE

Backing of the Hip-Rafter. At any convenient point in *be* (Fig. 9 draw *mn* at right-angles to *be*. From *o* describe a circle, tangent to *bh*, *bc* in *s*. Join *m* and *s* and *n* and *s*. The lines *ms* and *ns* form at *s* the angle for beveling the top of the hip-rafter.

5. TRIGONOMETRY

It is not the purpose of the author to teach the principles or uses of tri etry; but for the benefit of those readers who have already acquired a kno of this science, the following convenient formulas and tables of natura cosines, tangents and cotangents have been inserted. To those who kno to apply these trigonometric functions, they will often be found of gre venience and utility. These tables are taken, by permission, from Searle' Engineering, John Wiley & Sons, Inc., publishers.

Trigonometric Functions

A (Fig. 93) = angle BAC = arc BF and let the radius $AF = AB = AH = 1$ then

$\sin A = BC$

$\cos A = AC$

$\tan A = DF$

$\cot A = HG$

$\sec A = AD$

$\operatorname{cosec} A = AG$

$\operatorname{versin} A = CF = BE$

$\operatorname{covers} A = BK = HL$

$\operatorname{exsec} A = BD$

$\operatorname{coexsec} A = BG$

$\operatorname{chord} A = BF$

$\operatorname{chord} 2A = BI = 2\,BC$

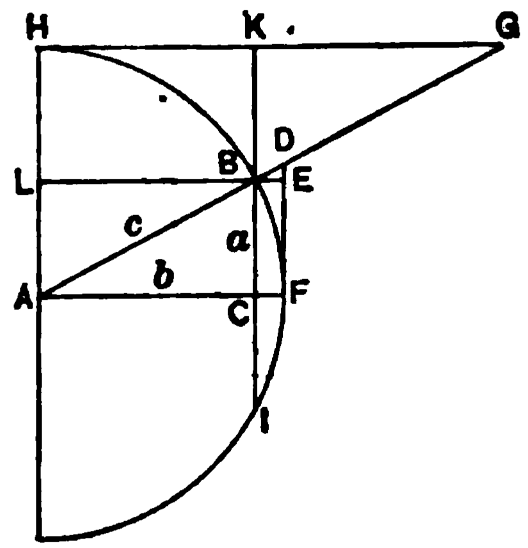

Fig. 93. Functions of Right-angled Triangle

the right-angled triangle ABC (Fig. 93) let $AB = c$, $AC = b$ and $BC = a$

$$\sin A = \frac{a}{c} = \cos B$$

$$\cos A = \frac{b}{c} = \sin B$$

$$\tan A = \frac{a}{b} = \cot B$$

$$\cot A = \frac{b}{a} = \tan B$$

$$\sec A = \frac{c}{b} = \operatorname{cosec} B$$

$$\operatorname{cosec} A = \frac{c}{a} = \sec B$$

$$\operatorname{vers} A = \frac{c-b}{c} = \operatorname{covers} B$$

$$\operatorname{exsec} A = \frac{c-b}{b} = \operatorname{coexsec} B$$

$$\operatorname{covers} A = \frac{c-a}{c} = \operatorname{versin} B$$

$$\operatorname{coexsec} A = \frac{c-a}{a} = \operatorname{exsec} B$$

(11) $a = c \sin A = b \tan A$

(12) $b = c \cos A = a \cot A$

(13) $c = \dfrac{a}{\sin A} = \dfrac{b}{\cos A}$

(14) $a = c \cos B = b \cot B$

(15) $b = c \sin B = a \tan B$

(16) $c = \dfrac{a}{\cos B} = \dfrac{b}{\sin B}$

(17) $a = \sqrt{(c+b)\,(c-b)}$

(18) $b = \sqrt{(c+a)\,(c-a)}$

(19) $c = \sqrt{a^2 + b^2}$

(20) $C = 90° = A + B$

(21) $\text{area} = \dfrac{ab}{2}$

Solution of Oblique Triangles

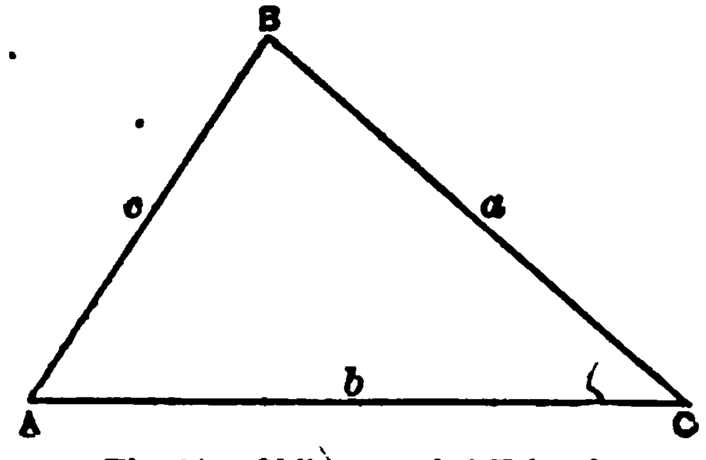

Fig. 94. Oblique-angled Triangle

	Given	Required	Formulas
(22)	A, B, a	C, b, c	$C = 180° - (A + B)$ $\qquad b = \dfrac{a}{\sin A} \cdot \sin B$ $c = \dfrac{a}{\sin A} \sin (A + B)$
(23)	A, a, b	B, C, c	$\sin B = \dfrac{\sin A}{a} \cdot b$ $\qquad C = 180° - (A + B$ $c = \dfrac{a}{\sin A} \cdot \sin C$
(24)	C, a, b	$\frac{1}{2}(A + B)$	$\frac{1}{2}(A + B) = 90° - \frac{1}{2}C$
(25)	....	$\frac{1}{2}(A - B)$	$\tan \frac{1}{2}(A - B) = \dfrac{a - b}{a + b} \tan \frac{1}{2}(A + B)$
(26)	....	A, B	$A = \frac{1}{2}(A + B) + \frac{1}{2}(A - B)$ $B = \frac{1}{2}(A + B) - \frac{1}{2}(A - B)$
(27)	....	c	$c = (a + b) \dfrac{\cos \frac{1}{2}(A + B)}{\cos \frac{1}{2}(A - B)} = (a - b) \dfrac{\sin \frac{1}{2}(A}{\sin \frac{1}{2}(A}$
(28)	....	Area	$K = \frac{1}{2} ab \sin C$
(29)	a, b, c	A	Let $s = \frac{1}{2}(a + b + c)$; $\quad \sin \frac{1}{2}A = \sqrt{\dfrac{(s - b)(}{bc}}$
(30)	....	....	$\cos \frac{1}{2}A = \sqrt{\dfrac{s(s - a)}{bc}}$; $\quad \tan \frac{1}{2}A = \sqrt{\dfrac{(s - b)(}{s(s -}}$
(31)	....	....	$\sin A = \dfrac{2\sqrt{s(s - a)(s - b)(s - c)}}{bc}$ $\text{vers } A = \dfrac{2(s - b)(s - c)}{bc}$
(32)	....	Area	$K = \sqrt{s(s - a)(s - b)(s - c)}$
(33)	A, B, C, a	Area	$K = \dfrac{a^2 \sin B \sin C}{2 \sin A}$

Oblique Triangles. General Formulas

$$\sin A = \frac{1}{\operatorname{cosec} A} = \sqrt{1-\cos^2 A} = \tan A \cos A$$

$$\sin A = 2 \sin \tfrac{1}{2} A \cos \tfrac{1}{2} A = \operatorname{vers} A \cot \tfrac{1}{2} A$$

$$\sin A = \sqrt{\tfrac{1}{2} \operatorname{vers} 2A} = \sqrt{\tfrac{1}{2}(1-\cos 2A)}$$

$$\cos A = \frac{1}{\sec A} = \sqrt{1-\sin^2 A} = \cot A \sin A$$

$$\cos A = 1 - \operatorname{vers} A = 2\cos^2 \tfrac{1}{2} A - 1 = 1 - 2\sin^2 \tfrac{1}{2} A$$

$$\cos A = \cos^2 \tfrac{1}{2} A - \sin^2 \tfrac{1}{2} A = \sqrt{\tfrac{1}{2} + \tfrac{1}{2}\cos 2A}$$

$$\tan A = \frac{1}{\cot A} = \frac{\sin A}{\cos A} = \sqrt{\sec^2 A - 1}$$

$$\tan A = \sqrt{\frac{1}{\cos^2 A} - 1} = \frac{\sqrt{1-\cos^2 A}}{\cos A} = \frac{\sin 2A}{1+\cos 2A}$$

$$\tan A = \frac{1-\cos 2A}{\sin 2A} = \frac{\operatorname{vers} 2A}{\sin 2A} = \operatorname{exsec} A \cot \tfrac{1}{2} A$$

$$\cot A = \frac{1}{\tan A} = \frac{\cos A}{\sin A} = \sqrt{\operatorname{cosec}^2 A - 1}$$

$$\cot A = \frac{\sin 2A}{1-\cos 2A} = \frac{\sin 2A}{\operatorname{vers} 2A} = \frac{1+\cos 2A}{\sin 2A}$$

$$\cot A = \frac{\tan \tfrac{1}{2} A}{\operatorname{exsec} A}$$

$$\operatorname{vers} A = 1 - \cos A = \sin A \tan \tfrac{1}{2} A = 2\sin^2 \tfrac{1}{2} A$$

$$\operatorname{vers} A = \operatorname{exsec} A \cos A$$

$$\operatorname{exsec} A = \sec A - 1 = \tan A \tan \tfrac{1}{2} A = \frac{\operatorname{vers} A}{\cos A}$$

$$\sin \tfrac{1}{2} A = \sqrt{\frac{1-\cos A}{2}} = \sqrt{\frac{\operatorname{vers} A}{2}}$$

$$\sin 2A = 2 \sin A \cos A$$

$$\cos \tfrac{1}{2} A = \sqrt{\frac{1+\cos A}{2}}$$

$$\cos 2A = 2\cos^2 A - 1 = \cos^2 A - \sin^2 A = 1 - 2\sin^2 A$$

$$\tan \tfrac{1}{2} A = \frac{\tan A}{1+\sec A} = \operatorname{cosec} A - \cot A = \frac{1-\cos A}{\sin A} = \sqrt{\frac{1-\cos A}{1+\cos A}}$$

$$\tan 2A = \frac{2\tan A}{1-\tan^2 A}$$

$$\cot \tfrac{1}{2} A = \frac{\sin A}{\operatorname{vers} A} = \frac{1+\cos A}{\sin A} = \frac{1}{\operatorname{cosec} A - \cot A}$$

$$\cot 2A = \frac{\cot^2 A - 1}{2 \cot A}$$

(57) $\operatorname{vers} \frac{1}{2} A = \dfrac{\frac{1}{2} \operatorname{vers} A}{1 + \sqrt{1 - \frac{1}{2} \operatorname{vers} A}} = \dfrac{1 - \cos A}{2 + \sqrt{2(1 + \cos A)}}$

(58) $\operatorname{vers} 2A = 2 \sin^2 A$

(59) $\operatorname{exsec} \frac{1}{2} A = \dfrac{1 - \cos A}{(1 + \cos A) + \sqrt{2(1 + \cos A)}}$

(60) $\operatorname{exsec} 2A = \dfrac{2 \tan^2 A}{1 - \tan^2 A}$

(61) $\sin (A \pm B) = \sin A \cos B \pm \sin B \cos A$

(62) $\cos (A \pm B) = \cos A \cos B \mp \sin A \sin B$

(63) $\sin A + \sin B = 2 \sin \frac{1}{2} (A + B) \cos \frac{1}{2} (A - B)$

(64) $\sin A - \sin B = 2 \cos \frac{1}{2} (A + B) \sin \frac{1}{2} (A - B)$

(65) $\cos A + \cos B = 2 \cos \frac{1}{2} (A + B) \cos \frac{1}{2} (A - B)$

(66) $\cos B - \cos A = 2 \sin \frac{1}{2} (A + B) \sin \frac{1}{2} (A - B)$

(67) $\sin^2 A - \sin^2 B = \cos^2 B - \cos^2 A = \sin (A + B) \sin (A - B)$

(68) $\cos^2 A - \sin^2 B = \cos (A + B) \cos (A - B)$

(69) $\tan A + \tan B = \dfrac{\sin (A + B)}{\cos A \cos B}$

(70) $\tan A - \tan B = \dfrac{\sin (A - B)}{\cos A \cos B}$

Table of Natural Sines and Cosines

′	15° Sine	15° Cosin	16° Sine	16° Cosin	17° Sine	17° Cosin	18° Sine	18° Cosin	19° Sine	19° Co
0	.25882	.96593	.27564	.96126	.29237	.95630	.30902	.95106	.32557	.94
1	.25910	.96585	.27592	.96118	.29265	.95622	.30929	.95097	.32584	.94
2	.25933	.96578	.27620	.96110	.29293	.95613	.30957	.95088	.32612	.94
3	.25966	.96570	.27648	.96102	.29321	.95605	.30985	.95079	.32639	.94
4	.25994	.96562	.27676	.96094	.29348	.95596	.31012	.95070	.32667	.94
5	.26022	.96555	.27704	.96086	.29376	.95588	.31040	.95061	.32694	.94
6	.26050	.96547	.27731	.96078	.29404	.95579	.31068	.95052	.32722	.94
7	.26079	.96540	.27759	.96070	.29432	.95571	.31095	.95043	.32749	.94
8	.26107	.96532	.27787	.96062	.29460	.95562	.31123	.95033	.32777	.94
9	.26135	.96524	.27815	.96054	.29487	.95554	.31151	.95024	.32804	.94
10	.26163	.96517	.27843	.96046	.29515	.95545	.31178	.95015	.32832	.94
11	.26191	.96509	.27871	.96037	.29543	.95536	.31206	.95006	.32859	.94
12	.26219	.96502	.27899	.96029	.20571	.95528	.31233	.94997	.32887	.94
13	.26247	.96494	.27927	.96021	.20599	.95510	.31261	.94988	.32914	.94
14	.26275	.96486	.27955	.96013	.20626	.95511	.31289	.94979	.32942	.94
15	.26303	.96479	.27983	.96005	.20654	.95502	.31316	.94970	.32969	.94
16	.26331	.96471	.28011	.95997	.20682	.95493	.31344	.94961	.32997	.94
17	.26359	.96463	.23039	.95989	.20710	.95485	.31372	.94952	.33024	.94
18	.26387	.96456	.23067	.95981	.30737	.95476	.31399	.94943	.33051	.94
19	.26415	.96448	.28095	.95972	.20765	.95467	.31427	.94933	.33079	.94
20	.26443	.96440	.28123	.95964	.29793	.95459	.31454	.94924	.33106	.94
21	.26471	.96433	.28150	.95956	.29821	.95450	.31482	.94915	.33134	.94
22	.26500	.96425	.23178	.95948	.20349	.95441	.31510	.94906	.33161	.94
23	.26528	.96417	.28206	.95940	.20376	.95433	.31537	.94897	.33189	.94
24	.26556	.96410	.28234	.95931	.20904	.95424	.31565	.94888	.33216	.94
25	.26584	.96402	.28262	.95923	.20932	.95415	.31593	.94878	.33244	.94
26	.26612	.96394	.28290	.95915	.29960	.95407	.31620	.94869	.33271	.94
27	.26640	.96386	.28318	.95907	.29987	.95398	.31648	.94860	.33298	.94
28	.26668	.96379	.28346	.95898	.30015	.95389	.31675	.94851	.33326	.94
29	.26696	.96371	.28374	.95890	.30043	.95380	.31703	.94842	.33353	.94
30	.26724	.96363	.28402	.95882	.30071	.95372	.31730	.94832	.33381	.94
31	.26752	.96355	.28429	.95874	.30098	.95363	.31758	.94823	.33408	.94
32	.26780	.96347	.28457	.95865	.30126	.95354	.31780	.94814	.33436	.94
33	.26808	.96340	.28485	.95857	.30154	.95345	.31813	.94805	.33463	.94
34	.26836	.96332	.28513	.95849	.30182	.95337	.31841	.94795	.33490	.94
35	.26864	.96324	.28541	.95841	.30209	.95328	.31868	.94786	.33518	.94
36	.26892	.96316	.28569	.95832	.30237	.95319	.31896	.94777	.33545	.94
37	.26920	.96308	.28597	.95824	.30265	.95310	.31923	.94768	.33573	.94
38	.26948	.96301	.28625	.95816	.30292	.95301	.31951	.94758	.33600	.94
39	.26076	.96293	.28652	.95807	.30320	.95293	.31979	.94749	.33627	.94
40	.27004	.96285	.28680	.95799	.30348	.95284	.32006	.94740	.33655	.94
41	.27032	.96277	.28708	.95791	.30376	.95275	.32034	.94730	.33682	.94
42	.27060	.96269	.28736	.95782	.30403	.95260	.32061	.94721	.33710	.94
43	.27088	.96261	.28764	.95774	.30431	.95257	.32080	.94712	.23737	.94
44	.27116	.96253	.28792	.95766	.30459	.95248	.32116	.94702	.33764	.94
45	.27144	.96246	.28820	.95757	.30486	.95240	.32144	.94693	.33792	.94
46	.27172	.96238	.28847	.95749	.30514	.95231	.32171	.94684	.33819	.94
47	.27200	.96230	.28875	.95740	.30542	.95222	.32199	.94674	.33846	.94
48	.27228	.96222	.28903	.95732	.30570	.95213	.32227	.94665	.33874	.94
49	.27256	.96214	.28931	.95724	.30597	.95204	.32254	.94656	.33901	.94
50	.27284	.96206	.28959	.95715	.30625	.95195	.32282	.94646	.33929	.94
51	.27312	.96198	.28987	.95707	.30653	.95186	.32309	.94637	.33956	.94
52	.27340	.96190	.29015	.95698	.30680	.95177	.32337	.94627	.33983	.94
53	.27368	.96182	.29042	.95690	.30708	.95168	.32364	.94618	.34011	.94
54	.27396	.96174	.29070	.95681	.30736	.95159	.32392	.94609	.34038	.94
55	.27424	.96166	.29098	.95673	.30763	.95150	.32419	.94599	.34065	.94
56	.27452	.96158	.29126	.95664	.30791	.95142	.32447	.94590	.24093	.94
57	.27480	.96150	.29154	.95656	.30819	.95133	.32474	.93580	.34120	.93
58	.27508	.96142	.29182	.95647	.30846	.95124	.32502	.94571	.34147	.93
59	.27536	.96134	.29209	.95639	.30874	.95115	.32529	.94561	.34175	.93
60	.27564	.96126	.29237	.95630	.30902	.95106	.32557	.94552	.34202	.93
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	S
	74°		73°		72°		71°		70°	

′	25°		26°		27°		28°		29°	
	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Co
0	.42262	.90631	.43837	.89879	.45399	.89101	.46947	.88295	.48481	.874
1	.42288	.90618	.43863	.89867	.45425	.89087	.46973	.88281	.48506	.874
2	.42315	.90606	.43889	.89854	.45451	.89074	.46999	.88267	.48532	.874
3	.42341	.90594	.43916	.89841	.45477	.89061	.47024	.88254	.48557	.874
4	.42367	.90582	.43942	.89828	.45503	.89048	.47050	.88240	.48583	.874
5	.42394	.90569	.43968	.89816	.45529	.89035	.47076	.88226	.48608	.873
6	.42420	.90557	.43994	.89803	.45554	.89021	.47101	.88213	.48634	.873
7	.42446	.90545	.44020	.89790	.45580	.89008	.47127	.88199	.48659	.873
8	.42473	.90532	.44046	.89777	.45606	.88995	.47153	.88185	.48684	.873
9	.42499	.90520	.44072	.89764	.45632	.88981	.47178	.88172	.48710	.873
10	.42525	.90507	.44098	.89752	.45658	.88968	.47204	.88158	.48735	.873
11	.42552	.90495	.44124	.89739	.45684	.88955	.47229	.88144	.48761	.873
12	.42578	.90483	.44151	.89726	.45710	.88942	.47255	.88130	.48786	.872
13	.42604	.90470	.44177	.89713	.45736	.88928	.47281	.88117	.48811	.872
14	.42631	.90458	.44203	.89700	.45762	.88915	.47306	.88103	.48837	.872
15	.42657	.90446	.44229	.89687	.45787	.88902	.47332	.88089	.48862	.872
16	.42683	.90433	.44255	.89674	.45813	.88888	.47358	.88075	.48888	.872
17	.42709	.90421	.44281	.89662	.45839	.88875	.47383	.88062	.48913	.872
18	.42736	.90408	.44307	.89649	.45865	.88862	.47409	.88048	.48938	.872
19	.42762	.90396	.44333	.89636	.45891	.88848	.47434	.88034	.48964	.8719
20	.42788	.90383	.44359	.89623	.45917	.88835	.47460	.88020	.48989	.8717
21	.42815	.90371	.44385	.89610	.45942	.88822	.47486	.88006	.49014	.8716
22	.42841	.90358	.44411	.89597	.45968	.88808	.47511	.87993	.49040	.8715
23	.42867	.90346	.44437	.89584	.45994	.88795	.47537	.87979	.49065	.8713
24	.42894	.90334	.44464	.89571	.46020	.88782	.47562	.87965	.49090	.8712
25	.42920	.90321	.44490	.89558	.46046	.88768	.47588	.87951	.49116	.8710
26	.42946	.90309	.44516	.89545	.46072	.88755	.47614	.87937	.49141	.870
27	.42972	.90296	.44542	.89532	.46097	.88741	.47639	.87923	.49166	.8707
28	.42999	.90284	.44568	.89519	.46123	.88728	.47665	.87909	.49192	.8706
29	.43025	.90271	.44594	.89506	.46149	.88715	.47690	.87896	.49217	.8705
30	.43051	.90259	.44620	.89493	.46175	.88701	.47716	.87882	.49242	.870
31	.43077	.90246	.44646	.89480	.46201	.88688	.47741	.87868	.49268	.8702
32	.43104	.90233	.44672	.89467	.46226	.88674	.47767	.87854	.49293	.8700
33	.43130	.90221	.44698	.89454	.46252	.88661	.47793	.87840	.49318	.869
34	.43156	.90208	.44724	.89441	.46278	.88647	.47818	.87826	.49344	.8697
35	.43182	.90196	.44750	.89428	.46304	.88634	.47844	.87812	.49369	.8696
36	.43209	.90183	.44776	.89415	.46330	.88620	.47869	.87798	.49394	.8694
37	.43235	.90171	.44802	.89402	.46355	.88607	.47895	.87784	.49419	.8693
38	.43261	.90158	.44828	.89389	.46381	.88593	.47920	.87770	.49445	.8692
39	.43287	.90146	.44854	.89376	.46407	.88580	.47946	.87756	.49470	.8690
40	.43313	.90133	.44880	.89363	.46433	.88566	.47971	.87743	.49495	.869
41	.43340	.90120	.44906	.89350	.46458	.88553	.47997	.87729	.49521	.8687
42	.43366	.90108	.44932	.89337	.46484	.88539	.48022	.87715	.49546	.8686
43	.43392	.90095	.44958	.89324	.46510	.88526	.48048	.87701	.49571	.8684
44	.43418	.90082	.44984	.89311	.46536	.88512	.48073	.87687	.49596	.8683
45	.43445	.90070	.45010	.89298	.46561	.88499	.48099	.87673	.49622	.8682
46	.43471	.90057	.45036	.89285	.46587	.88485	.48124	.87659	.49647	.8680
47	.43497	.90045	.45062	.89272	.46613	.88472	.48150	.87645	.49672	.8679
48	.43523	.90032	.45088	.89259	.46639	.88458	.48175	.87631	.49697	.8677
49	.43549	.90019	.45114	.89245	.46664	.88445	.48201	.87617	.49723	.8676
50	.43575	.90007	.45140	.89232	.46690	.88431	.48226	.87603	.49748	.8674
51	.43602	.89994	.45166	.89219	.46716	.88417	.48252	.87589	.49773	.8673
52	.43628	.89981	.45192	.89206	.46742	.88404	.48277	.87575	.49798	.8671
53	.43654	.89968	.45218	.89193	.46767	.88390	.48303	.87561	.49824	.8670
54	.43680	.89956	.45243	.89180	.46793	.88377	.48328	.87546	.49849	.8669
55	.43706	.89943	.45269	.89167	.46819	.88363	.48354	.87532	.49874	.8667
56	.43733	.89930	.45295	.89153	.46844	.88349	.48379	.87518	.49899	.8666
57	.43759	.89918	.45321	.89140	.46870	.88336	.48405	.87504	.49924	.8664
58	.43785	.89905	.45347	.89127	.46896	.88322	.48430	.87490	.49950	.8663
59	.43811	.89892	.45373	.89114	.46921	.88308	.48456	.87476	.49975	.8661
60	.43837	.89879	.45399	.89101	.46947	.88295	.48481	.87462	.50000	.8660
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine

Table of Natural Sines and Cosines

30°		31°		32°		33°		34°		
Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	′
[illegible]	.86603	.51504	.85717	.52992	.84805	.54464	.83867	.55919	.82904	60
[illegible]	.86588	.51529	.85702	.53017	.84789	.54488	.83851	.55943	.82887	59
[illegible]	.86573	.51554	.85687	.53041	.84774	.54513	.83835	.55968	.82871	58
[illegible]	.86559	.51579	.85672	.53066	.84759	.54537	.83819	.55992	.82855	57
[illegible]	.86544	.51604	.85657	.53091	.84743	.54561	.83804	.56016	.82839	56
[illegible]	.86530	.51628	.85642	.53115	.84728	.54586	.83788	.56040	.82822	55
[illegible]	.86515	.51653	.85627	.53140	.84712	.54610	.83772	.56064	.82806	54
[illegible]	.86501	.51678	.85612	.53164	.84697	.54635	.83756	.56088	.82790	53
[illegible]	.86486	.51703	.85597	.53189	.84681	.54659	.83740	.56112	.82773	52
[illegible]	.86471	.51728	.85582	.53214	.84666	.54683	.83724	.56136	.82757	51
[illegible]	.86457	.51753	.85567	.53238	.84650	.54708	.83708	.56160	.82741	50
[illegible]	.86442	.51778	.85551	.53263	.84635	.54732	.83692	.56184	.82724	49
[illegible]	.86427	.51803	.85536	.53288	.84619	.54756	.83676	.56208	.82708	48
[illegible]	.86413	.51828	.85521	.53312	.84604	.54781	.83660	.56232	.82692	47
[illegible]	.86398	.51852	.85506	.53337	.84588	.54805	.83645	.56256	.82675	46
[illegible]	.86384	.51877	.85491	.53361	.84573	.54829	.83629	.56280	.82659	45
[illegible]	.86369	.51902	.85476	.53386	.84557	.54854	.83613	.56305	.82643	44
[illegible]	.86354	.51927	.85461	.53411	.84542	.54878	.83597	.56329	.82626	43
[illegible]	.86340	.51952	.85446	.53435	.84526	.54902	.83581	.56353	.82610	42
[illegible]	.86325	.51977	.85431	.53460	.84511	.54927	.83565	.56377	.82593	41
[illegible]	.86310	.52002	.85416	.53484	.84495	.54951	.83549	.56401	.82577	40
[illegible]	.86295	.52026	.85401	.53509	.84480	.54975	.83533	.56425	.82561	39
[illegible]	.86281	.52051	.85385	.53534	.84464	.54999	.83517	.56449	.82544	38
[illegible]	.86266	.52076	.85370	.53558	.84448	.55024	.83501	.56473	.82528	37
[illegible]	.86251	.52101	.85355	.53583	.84433	.55048	.83485	.56497	.82511	36
[illegible]	.86237	.52126	.85340	.53607	.84417	.55072	.83469	.56521	.82495	35
[illegible]	.86222	.52151	.85325	.53632	.84402	.55097	.83453	.56545	.82478	34
[illegible]	.86207	.52175	.85310	.53656	.84386	.55121	.83437	.56569	.82462	33
[illegible]	.86192	.52200	.85294	.53681	.84370	.55145	.83421	.56593	.82446	32
[illegible]	.86178	.52225	.85279	.53705	.84355	.55169	.83405	.56617	.82429	31
[illegible]	.86163	.52250	.85264	.53730	.84339	.55194	.83389	.56641	.82413	30
[illegible]	.86148	.52275	.85249	.53754	.84324	.55218	.83373	.56665	.82396	29
[illegible]	.86133	.52299	.85234	.53779	.84308	.55242	.83356	.56689	.82380	28
[illegible]	.86119	.52324	.85218	.53804	.84292	.55266	.83340	.56713	.82363	27
[illegible]	.86104	.52349	.85203	.53828	.84277	.55291	.83324	.56736	.82347	26
[illegible]	.86089	.52374	.85188	.53853	.84261	.55315	.83308	.56760	.82330	25
[illegible]	.86074	.52399	.85173	.53877	.84245	.55339	.83292	.56784	.82314	24
[illegible]	.86059	.52423	.85157	.53902	.84230	.55363	.83276	.56808	.82297	23
[illegible]	.86045	.52448	.85142	.53926	.84214	.55388	.83260	.56832	.82281	22
[illegible]	.86030	.52473	.85127	.53951	.84198	.55412	.83244	.56856	.82264	21
[illegible]	.86015	.52498	.85112	.53975	.84182	.55436	.83228	.56880	.82248	20
[illegible]	.86000	.52522	.85096	.54000	.84167	.55460	.83212	.56904	.82231	19
[illegible]	.85985	.52547	.85081	.54024	.84151	.55484	.83195	.56928	.82214	18
[illegible]	.85970	.52572	.85066	.54049	.84135	.55509	.83179	.56952	.82198	17
[illegible]	.85956	.52597	.85051	.54073	.84120	.55532	.83163	.56976	.82181	16
[illegible]	.85941	.52621	.85035	.54097	.84104	.55557	.83147	.57000	.82165	15
[illegible]	.85926	.52646	.85020	.54122	.84088	.55581	.83131	.57024	.82148	14
[illegible]	.85911	.52671	.85005	.54146	.84072	.55605	.83115	.57047	.82132	13
[illegible]	.85896	.52696	.84989	.54171	.84057	.55630	.83098	.57071	.82115	12
[illegible]	.85881	.52720	.84974	.54195	.84041	.55654	.83082	.57095	.82098	11
[illegible]	.85866	.52745	.84959	.54220	.84025	.55678	.83066	.57119	.82082	10
[illegible]	.85851	.52770	.84943	.54244	.84009	.55702	.83050	.57143	.82065	9
[illegible]	.85836	.52794	.84928	.54269	.83994	.55726	.83034	.57167	.82048	8
[illegible]	.85821	.52819	.84913	.54293	.83978	.55750	.83017	.57191	.82032	7
[illegible]	.85806	.52844	.84897	.54317	.83962	.55775	.83001	.57215	.82015	6
[illegible]	.85792	.52869	.84882	.54342	.83946	.55799	.82985	.57238	.81999	5
[illegible]	.85777	.52893	.84866	.54366	.83930	.55823	.82969	.57262	.81982	4
[illegible]	.85762	.52918	.84851	.54391	.83915	.55847	.82953	.57286	.81965	3
[illegible]	.85747	.52943	.84836	.54415	.83899	.55871	.82936	.57310	.81949	2
[illegible]	.85732	.52967	.84820	.54440	.83883	.55895	.82920	.57334	.81932	1
[illegible]	.85717	.52992	.84805	.54464	.83867	.55919	.82904	.57358	.81915	0
Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
59°		58°		57°		56°		55°		

′	35°		36°		37°		38°		39°	
	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosi
0	.57358	.81915	.58779	.80902	.60182	.79864	.61566	.78801	.62932	.777
1	.57381	.81899	.58802	.80885	.60205	.79846	.61589	.78783	.62955	.7769
2	.57405	.81882	.58826	.80867	.60228	.79829	.61612	.78765	.62977	.7767
3	.57429	.81865	.58849	.80850	.60251	.79811	.61635	.78747	.63000	.7766
4	.57453	.81848	.58873	.80833	.60274	.79793	.61658	.78729	.63022	.7764
5	.57477	.81832	.58896	.80816	.60298	.79776	.61681	.78711	.63045	.7762
6	.57501	.81815	.58920	.80799	.60321	.79758	.61704	.78694	.63068	.7760
7	.57524	.81798	.58943	.80782	.60344	.79741	.61726	.78676	.63090	.7758
8	.57548	.81782	.58967	.80765	.60367	.79723	.61749	.78658	.63113	.7756
9	.57572	.81765	.58990	.80748	.60390	.79706	.61772	.78640	.63135	.7755
10	.57596	.81748	.59014	.80730	.60414	.79688	.61795	.78622	.63158	.7753
11	.57619	.81731	.59037	.80713	.60437	.79671	.61818	.78604	.63180	.7751
12	.57643	.81714	.59061	.80696	.60460	.79653	.61841	.78586	.63203	.7749
13	.57667	.81698	.59084	.80679	.60483	.79635	.61864	.78568	.63225	.7747
14	.57691	.81681	.59108	.80662	.60506	.79618	.61887	.78550	.63248	.7745
15	.57715	.81664	.59131	.80644	.60529	.79600	.61909	.78532	.63271	.7743
16	.57738	.81647	.59154	.80627	.60553	.79583	.61932	.78514	.63293	.7742
17	.57762	.81631	.59178	.80610	.60576	.79565	.61955	.78496	.63316	.7740
18	.57786	.81614	.59201	.80593	.60599	.79547	.61978	.78478	.63338	.7738
19	.57810	.81597	.59225	.80576	.60622	.79530	.62001	.78460	.63361	.7736
20	.57833	.81580	.59248	.80558	.60645	.79512	.62024	.78442	.63383	.7734
21	.57857	.81563	.59272	.80541	.60668	.79494	.62046	.78424	.63406	.7732
22	.57881	.81546	.59295	.80524	.60691	.79477	.62069	.78405	.63428	.7731
23	.57904	.81530	.59318	.80507	.60714	.79459	.62092	.78387	.63451	.7729
24	.57928	.81513	.59342	.80489	.60738	.79441	.62115	.78369	.63473	.7727
25	.57952	.81496	.59365	.80472	.60761	.79424	.62138	.78351	.63496	.7725
26	.57976	.81479	.59389	.80455	.60784	.79406	.62160	.78333	.63518	.7723
27	.57999	.81462	.59412	.80438	.60807	.79388	.62183	.78315	.63540	.7721
28	.58023	.81445	.59436	.80420	.60830	.79371	.62206	.78297	.63563	.7719
29	.58047	.81428	.59459	.80403	.60853	.79353	.62229	.78279	.63585	.7718
30	.58070	.81412	.59482	.80386	.60876	.79335	.62251	.78261	.63608	.7716
31	.58094	.81395	.59506	.80368	.60899	.79318	.62274	.78243	.63630	.7714
32	.58118	.81378	.59529	.80351	.60922	.79300	.62297	.78225	.63653	.7712
33	.58141	.81361	.59552	.80334	.60945	.79282	.62320	.78206	.63675	.7710
34	.58165	.81344	.59576	.80316	.60968	.79264	.62342	.78188	.63698	.7708
35	.58189	.81327	.59599	.80299	.60991	.79247	.62365	.78170	.63720	.7707
36	.58212	.81310	.59622	.80282	.61015	.79229	.62388	.78152	.63742	.7705
37	.58236	.81293	.59646	.80264	.61038	.79211	.62411	.78134	.63765	.7703
38	.58260	.81276	.59669	.80247	.61061	.79193	.62433	.78116	.63787	.7701
39	.58283	.81259	.59693	.80230	.61084	.79176	.62456	.78098	.63810	.7699
40	.58307	.81242	.59716	.80212	.61107	.79158	.62479	.78079	.63832	.7697
41	.58330	.81225	.59739	.80195	.61130	.79140	.62502	.78061	.63854	.7695
42	.58354	.81208	.59763	.80178	.61153	.79122	.62524	.78043	.63877	.7694
43	.58378	.81191	.59786	.80160	.61176	.79105	.62547	.78025	.63899	.7692
44	.58401	.81174	.59809	.80143	.61199	.79087	.62570	.78007	.63922	.7690
45	.58425	.81157	.59832	.80125	.61222	.79069	.62592	.77988	.63944	.7688
46	.58449	.81140	.59856	.80108	.61245	.79051	.62615	.77970	.63966	.7686
47	.58472	.81123	.59879	.80091	.61268	.79033	.62638	.77952	.63989	.7684
48	.58496	.81106	.59902	.80073	.61291	.79016	.62660	.77934	.64011	.7682
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50	.58543	.81072	.59949	.80038	.61337	.78980	.62706	.77897	.64056	.7679
51	.58567	.81055	.59972	.80021	.61360	.78962	.62728	.77879	.64078	.7677
52	.58590	.81038	.59995	.80003	.61383	.78944	.62751	.77861	.64100	.7675
53	.58614	.81021	.60019	.79986	.61406	.78926	.62774	.77843	.64123	.7673
54	.58637	.81004	.60042	.79968	.61429	.78908	.62796	.77824	.64145	.7671
55	.58661	.80987	.60065	.79951	.61451	.78891	.62819	.77806	.64167	.7669
56	.58684	.80970	.60089	.79934	.61474	.78873	.62842	.77788	.64190	.7667
57	.58708	.80953	.60112	.79916	.61497	.78855	.62864	.77769	.64212	.7666
58	.58731	.80936	.60135	.79899	.61520	.78837	.62887	.77751	.64234	.7664
59	.58755	.80919	.60158	.79881	.61543	.78819	.62909	.77733	.64256	.7662
60	.58779	.80902	.60182	.79864	.61566	.78801	.62932	.77715	.64279	.7660
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0	.00000	Infinite	.01746	57.2900	[illegible]	[illegible]	[illegible]	[illegible]
1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	28.3994	[illegible]	[illegible]
2	.00058	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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5	[illegible]	687.540	[illegible]	52.9521	[illegible]	[illegible]	[illegible]	[illegible]
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11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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39	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	21.0056	[illegible]	[illegible]
40	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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44	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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47	.01367	73.1390	.03114	[illegible]	[illegible]	[illegible]	.06613	[illegible]
48	.01396	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
49	.01425	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
50	.01454	68.7501	[illegible]	31.2416	[illegible]	[illegible]	.06700	[illegible]
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58	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
59	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
60	.01746	57.2900	[illegible]	[illegible]	.05241	[illegible]	[illegible]	[illegible]
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2	.14113	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
13	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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27	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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39	.15213	[illegible]	[illegible]	[illegible]	.18906	[illegible]	[illegible]	[illegible]
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41	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
42	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang
	81°		80°		79°		78°	

Table of Natural Tangents and Cotangents

12° Tang	12° Cotang	13° Tang	13° Cotang	14° Tang	14° Cotang	15° Tang	15° Cotang
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4.01078	[illegible]	[illegible]
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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang
77°							

'	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang
0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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27	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
28	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
29	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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20°		21°				23°	
Tang	Cotang	Tang	Cotang			Tang	Cotang
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Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang
61°		60°		59°		58°	

′	Tang	Cotang	Tang	Cotang	Tang	Cotang	35° Tang	35° Cotang
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2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang

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Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang
[illegible]		[illegible]		[illegible]		[illegible]	

′	40° Tang	40° Cotang	41° Tang	41° Cotang	42° Tang	42° Cotang	Tang
0	.83910	1.19175	.86929	1.15037	.90040	1.11061	.9325
1	.83960	1.19105	.86980	1.14969	.90093	1.10996	.9330
2	.84009	1.19035	.87031	1.14902	.90146	1.10931	.9336
3	.84059	1.18964	.87082	1.14834	.90199	1.10867	.9341
4	.84108	1.18894	.87133	1.14767	.90251	1.10802	.9346
5	.84158	1.18824	.87184	1.14699	.90304	1.10737	.9352
6	.84208	1.18754	.87236	1.14632	.90357	1.10672	.9357
7	.84258	1.18684	.87287	1.14565	.90410	1.10607	.9363
8	.84307	1.18614	.87338	1.14498	.90463	1.10543	.9368
9	.84357	1.18544	.87389	1.14430	.90516	1.10478	.9374
10	.84407	1.18474	.87441	1.14363	.90569	1.10414	.9379
11	.84457	1.18404	.87492	1.14296	.90621	1.10349	.9385
12	.84507	1.18334	.87543	1.14229	.90674	1.10285	.9390
13	.84556	1.18264	.87595	1.14162	.90727	1.10220	.93961
14	.84606	1.18194	.87646	1.14095	.90781	1.10156	.9401
15	.84656	1.18125	.87698	1.14028	.90834	1.10091	.94071
16	.84706	1.18055	.87749	1.13961	.90887	1.10027	.9412
17	.84756	1.17986	.87801	1.13894	.90940	1.09963	.9418
18	.84806	1.17916	.87852	1.13828	.90993	1.09899	.9423
19	.84856	1.17846	.87904	1.13761	.91046	1.09834	.9429
20	.84906	1.17777	.87955	1.13694	.91099	1.09770	.9434
21	.84956	1.17708	.88007	1.13627	.91153	1.09706	.94400
22	.85006	1.17638	.88059	1.13561	.91206	1.09642	.94455
23	.85057	1.17569	.88110	1.13494	.91259	1.09578	.94510
24	.85107	1.17500	.88162	1.13428	.91313	1.09514	.94565
25	.85157	1.17430	.88214	1.13361	.91366	1.09450	.94620
26	.85207	1.17361	.88265	1.13295	.91419	1.09386	.94676
27	.85257	1.17292	.88317	1.13228	.91473	1.09322	.94731
28	.85308	1.17223	.88369	1.13162	.91526	1.09258	.94786
29	.85358	1.17154	.88421	1.13096	.91580	1.09195	.94841
30	.85408	1.17085	.88473	1.13029	.91633	1.09131	.94896
31	.85458	1.17016	.88524	1.12963	.91687	1.09067	.94952
32	.85509	1.16947	.88576	1.12897	.91740	1.09003	.95007
33	.85559	1.16878	.88628	1.12831	.91794	1.08940	.95062
34	.85609	1.16809	.88680	1.12765	.91847	1.08876	.95118
35	.85660	1.16741	.88732	1.12699	.91901	1.08813	.95173
36	.85710	1.16672	.88784	1.12633	.91955	1.08749	.95229
37	.85761	1.16603	.88836	1.12567	.92008	1.08686	.95284
38	.85811	1.16535	.88888	1.12501	.92062	1.08622	.95340
39	.85862	1.16466	.88940	1.12435	.92116	1.08559	.95395
40	.85912	1.16398	.88992	1.12369	.92170	1.08496	.95451
41	.85963	1.16329	.89045	1.12303	.92224	1.08432	.95506
42	.86014	1.16261	.89097	1.12238	.92277	1.08369	.95562
43	.86064	1.16192	.89149	1.12172	.92331	1.08306	.95618
44	.86115	1.16124	.89201	1.12106	.92385	1.08243	.95673
45	.86166	1.16056	.89253	1.12041	.92439	1.08179	.95729
46	.86216	1.15987	.89306	1.11975	.92493	1.08116	.95785
47	.86267	1.15919	.89358	1.11909	.92547	1.08053	.95841
48	.86318	1.15851	.89410	1.11844	.92601	1.07990	.95897
49	.86368	1.15783	.89463	1.11778	.92655	1.07927	.95952
50	.86419	1.15715	.89515	1.11713	.92709	1.07864	.96008
51	.86470	1.15647	.89567	1.11648	.92763	1.07801	.96064
52	.86521	1.15579	.89620	1.11582	.92817	1.07738	.96120
53	.86572	1.15511	.89672	1.11517	.92872	1.07676	.96176
54	.86623	1.15443	.89725	1.11452	.92926	1.07613	.96232
55	.86674	1.15375	.89777	1.11387	.92980	1.07550	.96288
56	.86725	1.15308	.89830	1.11321	.93034	1.07487	.96344
57	.86776	1.15240	.89883	1.11256	.93088	1.07425	.96400
58	.86827	1.15172	.89935	1.11191	.93143	1.07362	.96457
59	.86878	1.15104	.89988	1.11126	.93197	1.07299	.96513
60	.86929	1.15037	.90040	1.11061	.93252	1.07237	.96569
′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang
	49°		48°		47°		

44°		′	′	44°		′	′	44°	
Tang	Cotang			Tang	Cotang			Tang	Cota
.96569	1.03553	60	20	.97700	1.02355	40	40	.98843	1.011
.96625	1.03493	59	21	.97756	1.02295	39	41	.98901	1.011
.96681	1.03433	58	22	.97813	1.02236	38	42	.98958	1.010
.96738	1.03372	57	23	.97870	1.02176	37	43	.99016	1.009
.96794	1.03312	56	24	.97927	1.02117	36	44	.99073	1.009
.96850	1.03252	55	25	.97984	1.02057	35	45	.99131	1.008
.96907	1.03192	54	26	.98041	1.01998	34	46	.99189	1.008
.96963	1.03132	53	27	.98098	1.01939	33	47	.99247	1.007
.97020	1.03072	52	28	.98155	1.01879	32	48	.99304	1.007
.97076	1.03012	51	29	.98213	1.01820	31	49	.99362	1.006
.97133	1.02952	50	30	.98270	1.01761	30	50	.99420	1.005
.97189	1.02892	49	31	.98327	1.01702	29	51	.99478	1.005
.97246	1.02832	48	32	.98384	1.01642	28	52	.99536	1.004
.97302	1.02772	47	33	.98441	1.01583	27	53	.99594	1.004
.97359	1.02713	46	34	.98499	1.01524	26	54	.99652	1.003
.97416	1.02652	45	35	.98556	1.01465	25	55	.99710	1.002
.97472	1.02593	44	36	.98613	1.01406	24	56	.99768	1.002
.97529	1.02533	43	37	.98671	1.01347	23	57	.99826	1.001
.97586	1.02474	42	38	.98728	1.01288	22	58	.99884	1.001
.97643	1.02414	41	39	.98786	1.01229	21	59	.99942	1.000
.97700	1.02355	40	40	.98843	1.01170	20	60	1.00000	1.000
Cotang	Tang			Cotang	Tang			Cotang	Tan
45°		′	′	45°		′	′	45°	

Natural Secants and Cosecants

Degrees	Secants						
	0′	10′	20′	30′	40′	50′	60′
0	1.00000	1.00001	1.00002	1.00004	1.00007	1.00011	1.00015
1	1.00015	1.00021	1.00027	1.00034	1.00042	1.00051	1.00061
2	1.00061	1.00072	1.00083	1.00095	1.00108	1.00122	1.00137
3	1.00137	1.00153	1.00169	1.00187	1.00205	1.00224	1.00244
4	1.00244	1.00265	1.00287	1.00309	1.00333	1.00357	1.00382
5	1.00382	1.00408	1.00435	1.00463	1.00491	1.00521	1.00551
6	1.00551	1.00582	1.00614	1.00647	1.00681	1.00715	1.00751
7	1.00751	1.00787	1.00825	1.00863	1.00902	1.00942	1.00983
8	1.00983	1.01024	1.01067	1.01111	1.01155	1.01200	1.01247
9	1.01247	1.01294	1.01342	1.01391	1.01440	1.01491	1.01543
10	1.01543	1.01595	1.01649	1.01703	1.01758	1.01815	1.01872
11	1.01872	1.01930	1.01989	1.02049	1.02110	1.02171	1.02234
12	1.02234	1.02298	1.02362	1.02428	1.02494	1.02562	1.02630
13	1.02630	1.02700	1.02770	1.02842	1.02914	1.02987	1.03061
14	1.03061	1.03137	1.03213	1.03290	1.03368	1.03447	1.03528
15	1.03528	1.03609	1.03691	1.03774	1.03858	1.03944	1.04030
16	1.04030	1.04117	1.04206	1.04295	1.04385	1.04477	1.04569
17	1.04569	1.04663	1.04757	1.04853	1.04950	1.05047	1.05146
18	1.05146	1.05246	1.05347	1.05449	1.05552	1.05657	1.05762
19	1.05762	1.05869	1.05976	1.06085	1.06195	1.06306	1.06418
20	1.06418	1.06531	1.06645	1.06761	1.06878	1.06995	1.07115
21	1.07115	1.07235	1.07356	1.07479	1.07602	1.07727	1.07853
22	1.07853	1.07981	1.08109	1.08239	1.08370	1.08503	1.08636
23	1.08636	1.08771	1.08907	1.09044	1.09183	1.09323	1.09464
24	1.09464	1.09606	1.09750	1.09895	1.10041	1.10189	1.10338
25	1.10338	1.10488	1.10640	1.10793	1.10947	1.11103	1.11260
26	1.11260	1.11419	1.11579	1.11740	1.11903	1.12067	1.12233
27	1.12233	1.12400	1.12568	1.12738	1.12910	1.13083	1.13257
28	1.13257	1.13433	1.13610	1.13789	1.13970	1.14152	1.14335
29	1.14335	1.14521	1.14707	1.14896	1.15085	1.15277	1.15470
30	1.15470	1.15665	1.15861	1.16059	1.16259	1.16460	1.16663
31	1.16663	1.16868	1.17075	1.17283	1.17493	1.17704	1.17918
32	1.17918	1.18133	1.18350	1.18569	1.18790	1.19012	1.19236
33	1.19236	1.19463	1.19691	1.19920	1.20152	1.20386	1.20622
34	1.20622	1.20859	1.21099	1.21341	1.21584	1.21830	1.22077
35	1.22077	1.22327	1.22579	1.22833	1.23089	1.23347	1.23607
36	1.23607	1.23869	1.24134	1.24400	1.24669	1.24940	1.25214
37	1.25214	1.25489	1.25767	1.26047	1.26330	1.26615	1.26902
38	1.26902	1.27191	1.27483	1.27778	1.28075	1.28374	1.28676
39	1.28676	1.28980	1.29287	1.29597	1.29909	1.30223	1.30541
40	1.30541	1.30861	1.31183	1.31509	1.31837	1.32168	1.32501
41	1.32501	1.32838	1.33177	1.33519	1.33864	1.34212	1.34563
42	1.34563	1.34917	1.35274	1.35634	1.35997	1.36363	1.36733
43	1.36733	1.37105	1.37481	1.37860	1.38242	1.38628	1.39016
44	1.39016	1.39409	1.39804	1.40203	1.40606	1.41012	1.41421
	60′	50′	40′	30′	20′	10′	0′
	Cosecants						

Natural Secants and Cosecants (Continued)

Cosecants							
0′	10′	20′	30′	40′	50′	60′	
∞	343.77516	171.88831	114.59301	85.94561	68.75736	57.29869	89
57.29869	49.11406	42.97571	38.20155	34.38232	31.25758	28.65371	88
28.65371	26.45051	24.56212	22.92559	21.49368	20.23028	19.10732	87
19.10732	18.10262	17.19843	16.38041	15.63679	14.95788	14.33559	86
14.33559	13.76312	13.23472	12.74550	12.29125	11.86837	11.47371	85
11.47371	11.10455	10.75849	10.43343	10.12752	9.83912	9.56677	84
9.56677	9.30917	9.06515	8.83367	8.61379	8.40466	8.20551	83
8.20551	8.01565	7.83443	7.66130	7.49571	7.33719	7.18530	82
7.18530	7.03962	6.89979	6.76547	6.63633	6.51208	6.39245	81
6.39245	6.27719	6.16607	6.05886	5.95536	5.85539	5.75877	80
5.75877	5.66533	5.57493	5.48740	5.40263	5.32049	5.24084	79
5.24084	5.16359	5.08863	5.01585	4.94517	4.87649	4.80973	78
4.80973	4.74482	4.68167	4.62023	4.56041	4.50216	4.44541	77
4.44541	4.39012	4.33622	4.28366	4.23239	4.18238	4.13357	76
4.13357	4.08591	4.03938	3.99393	3.94952	3.90613	3.86370	75
3.86370	3.82223	3.78166	3.74198	3.70315	3.66515	3.62796	74
3.62796	3.59154	3.55587	3.52094	3.48671	3.45317	3.42030	73
3.42030	3.38808	3.35649	3.32551	3.29512	3.26531	3.23607	72
3.23607	3.20737	3.17920	3.15155	3.12440	3.09774	3.07155	71
3.07155	3.04584	3.02057	2.99574	2.97135	2.94737	2.92380	70
2.92380	2.90063	2.87785	2.85545	2.83342	2.81175	2.79043	69
2.79043	2.76945	2.74881	2.72850	2.70851	2.68884	2.66947	68
2.66947	2.65040	2.63162	2.61313	2.59491	2.57698	2.55930	67
2.55930	2.54190	2.52474	2.50784	2.49119	2.47477	2.45859	66
2.45859	2.44264	2.42692	2.41142	2.39614	2.38107	2.36620	65
2.36620	2.35154	2.33708	2.32282	2.30875	2.29487	2.28117	64
2.28117	2.26766	2.25432	2.24116	2.22817	2.21535	2.20269	63
2.20269	2.19019	2.17786	2.16568	2.15366	2.14178	2.13005	62
2.13005	2.11847	2.10704	2.09574	2.08458	2.07356	2.06267	61
2.06267	2.05191	2.04128	2.03077	2.02039	2.01014	2.00000	60
2.00000	1.98998	1.98008	1.97029	1.96062	1.95106	1.94160	59
1.94160	1.93226	1.92302	1.91388	1.90485	1.89591	1.88708	58
1.88708	1.87834	1.86990	1.86116	1.85271	1.84435	1.83608	57
1.83608	1.82790	1.81981	1.81180	1.80388	1.79604	1.78829	56
1.78829	1.78062	1.77303	1.76552	1.75808	1.75073	1.74345	55
1.74345	1.73624	1.72911	1.72205	1.71506	1.70815	1.70130	54
1.70130	1.69452	1.68782	1.68117	1.67460	1.66809	1.66164	53
1.66164	1.65526	1.64894	1.64268	1.63648	1.63035	1.62427	52
1.62427	1.61825	1.61229	1.60639	1.60054	1.59475	1.58902	51
1.58902	1.58333	1.57771	1.57213	1.56661	1.56114	1.55572	50
1.55572	1.55036	1.54504	1.53977	1.53455	1.52938	1.52425	49
1.52425	1.51918	1.51415	1.50916	1.50422	1.49933	1.49448	48
1.49448	1.48967	1.48491	1.48019	1.47551	1.47087	1.46628	47
1.46628	1.46173	1.45721	1.45274	1.44831	1.44391	1.43956	46
1.43956	1.43524	1.43096	1.42672	1.42251	1.41835	1.41421	45
60′	50′	40′	30′	20′	10′	0′	Degrees
Secants							

PART II

RENGTH OF MATERIALS AND STABILITY OF STRUCTURES

INTRODUCTION

LANATION OF SUBJECT-MATTER AND NOTATION

1. Introduction to Part II

ject-Matter of Part II. In the thirty-one chapters of Part II are the necessary rules, formulas and data for computing the strength and ity of all ordinary forms of building-construction, whether of wood, steel, te or masonry, and in fact of all but the more intricate problems of steel uction, with which few architects care to cope, and which, indeed, are especially within the province of the engineer.

e Rules and Formulas have been reduced to their simplest forms, and, eral, require only an elementary knowledge of mathematics to understand . The application of the formulas is explained and in most cases their ation, and it is believed that the formulas, constants and working stresses epresentative of conservative and approved contemporary practice.

nstants and Working Stresses. In the use of constants for the strength terials, the authors and editors have been guided by the practice of leading tural engineers, by the available records of tests and by their own expe- e of many years as practicing and consulting architects and engineers. arying conditions of building-construction have been taken into account n attempt made to adapt the values to the practical conditions usually ning such construction. Every possible precaution has been taken to nt the misapplication of rules and formulas and to insure absolute safety ut undue waste of materials.

bles. Much thought and labor have been expended on the preparation e numerous tables, to insure their accuracy and to arrange them in the convenient form for use by architects and builders. Many of these tables computed by the authors and editors, all have been carefully verified, and believed that they may be used with perfect confidence. In all cases, un- therwise noted, they give the same values that would be obtained by g the formulas specially referred to, while they afford a great saving of and labor and reduce to a minimum the danger of errors in making the ary computations.

atment of the Subject. Owing to the nature of the subjects treated he large number of pages required to include them all in one book of refer- some forms of construction are treated rather briefly. The intention is to the data needed for immediate use rather than a complete discussion of e principles involved. Those who wish more complete discussions of s' work, carpenters' work, steelwork, etc., are referred to treatises on branches of building-construction. References are made in the different ers to various other books and periodicals containing more complete ation on some of the subjects. The thirty-one chapters of Part II principally with foundations, walls and piers, arches, columns, beams girders, floors, mill-construction, fireproofing, reinforced-concrete con- tion, roof-trusses, wind-bracing, and domes and vaults.

2. Explanation of the Notation or Symbols used in Part

Besides the usual mathematical signs and characters in general use lowing abbreviations and symbols are frequently used:

- A area of cross-section; also, a constant used in Chapter XVI a to 1/18 the safe unit fiber-stress;
- $a, b, c, \ldots m$, etc., known or given distances;
- b breadth, as of beams;
- C coefficient of strength;
- c normal distance from neutral axis of cross-section of beam distant fiber in same;
- d diameter, as of rivets; exterior diameter; depth, as of beams;
- d_1 interior diameter;
- E modulus of elasticity;
- E_s, E_c modulus of elasticity for steel and concrete respectively (inforced concrete);
- e total deformation or change in length, as in a bar;
- F shearing-modulus of elasticity;
- f maximum deflection for a beam;
- h distance between parallel axes for moments of inertia;
- I moment of inertia about a line;
- I/c section-modulus or section-factor;
- J polar moment of inertia;
- J' polar moment of inertia of bolts about shaft-axis;
- K total elastic resistance of a bar; resilience, work; also, a facto stant used in formulas for reinforced concrete;
- l length; span of a beam;
- M bending moment;
- M_{max} maximum bending moment;
- M_1, M_2, etc., bending moments at supports of beams;
- M_r or SI/c moment of resistance;
- n number of loads, spans, etc.;
- P external force; concentrated load;
- P_1, P_2, P_3, etc., concentrated loads on beams;
- p pitch of rivets; eccentricity of load on column; ratio of cros of steel to cross-section of beam (reinforced concrete);
- r radius of curvature; radius; radius of gyration; ratio of E_s to E_c for concrete (reinforced concrete);
- R_1, R_2, R_3, etc., reactions at the supports of a beam;
- S unit stress, with subscripts t, c and s for unit stress in tensi pression and shear, respectively;
- S_b buckling resistance in webs of steel beams;
- S_h horizontal unit shearing-stress in beams;
- S_e elastic limit;
- S_f modulus of rupture, or computed flexural strength;
- t_1, t_2, etc., thicknesses;
- V vertical shear;
- W weight of a bar or beam; total uniform load on beam (may weight of beam);
- wl total uniform load on a beam (may include weight of beam);
- w weight of a cubic unit of material; uniform load on beam, p unit of length;

* See, also, page 3 of Part I.

x, y, z, variable distances;
a, b, etc., material constants;
ϕ constant depending upon material;
θ an angle.

[Gre]ek letters are used generally for signs of operation, for abstract numbers [or] for angles. Σ is employed as a symbol of summation.

[Th]e following are the Greek letters most in use:

α Alpha,	β Beta,	ϵ Epsilon,	η Eta,
θ Theta,	κ Kappa,	λ Lambda,	μ Mu,
ν Nu,	π Pi,	ρ Rho,	σ Sigma,
τ Tau,	ϕ Phi,	ψ Psi,	ω Omega.

[Not]e. In a few places in the book it has been considered necessary or advis[able] by some of the associate editors to give a different meaning to one or [more] of the above symbols or to introduce different symbols for the meanings [give]n in the list, but in all such cases the new symbols or meanings have been [c]learly indicated.

[Th]e term BREADTH is used to denote the horizontal thickness of a beam or [the s]maller dimension of the cross-section of a rectangular column, post or [tie], and is always measured in inches unless expressly stated otherwise.

[Th]e term DEPTH denotes the vertical height of a beam or girder, and is always [measur]ed in inches unless expressly stated otherwise.

[Th]e term LENGTH denotes the distance between supports and is always meas[ured i]n feet unless expressly stated otherwise.

[Ab]breviations. In order to shorten the formulas, the tabulations of computa[tions], etc., and throughout the text generally, to economize space, the units of [measu]rement are generally abbreviated. For example, foot and feet are abbre[viate]d ft; inch and inches, in; pound and pounds, lb; square, sq; cubic, cu; [...]; in; inch-pound or inch-pounds, in-lb; foot-pound or foot-pounds, ft-lb; [ounce]s, oz; horse-power, h.p.; gallons, gal; etc.; and no periods are placed after [the] abbreviations, except at the ends of sentences. Where the word TON is [used i]n this volume, it always means the net ton of 2 000 lb.

CHAPTER I

EXPLANATION OF TERMS USED IN ARCHITECT ENGINEERING

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1. Definitions of Some of the Terms Used in the Mechani Materials *

Terms Used in Architectural Engineering. The following te quently occur in discussions of the principles of architectural engineer an understanding of their meaning is essential.

Mechanics is the branch of physics that treats of the phenomena ca the action of forces on material bodies.

Applied Mechanics treats of the laws of mechanics as applied to c tion in the useful arts, as in beams, trusses, arches, etc.

Mechanics of Materials treats of the effects of forces in causing in the size and shape of bodies.

Rest is the relation that exists between two points when the strai joining them does not change in length or direction. A body is at rest re to a point when any point in the body is at rest relatively to the first-me point.

Motion is the relation that exists between two points when the strai joining them changes in length or direction, or in both. A body mov tively to a point when any point in the body moves relatively to the fir tioned point.

Force is that which changes, or tends to change, the state of rest or of the body acted upon. It is a cause regarding the essential nature o we are ignorant. In the mechanics of materials we do not deal with the of forces, but only with the laws of their action.

Equilibrium is that condition of a body in which the forces acting balance or neutralize each other; or, it is that condition of a force-sy which the resultant of the force-system is zero.

Statics is the branch of Mechanics that treats of the conditions of equil It is divided into:

(1) Statics of rigid bodies.

(2) Statics of practically incompressible fluids.

In building-construction we have to deal only with the former.

Structures are artificial constructions in which all the parts are inte be in equilibrium and at rest relatively to each other, as in the case of a truss or roof-truss. They consist of two or more solid bodies, generally PIECES or MEMBERS, which are connected at different parts of their s called JOINTS.

* In addition to the terms defined here, many others are defined in the cha Part II, and especially in Chapters VI, IX, X, XIV, XV, XVI, XX and XXIV.

general there are three conditions of equilibrium in a structure.

(1) The external forces acting upon the whole structure must balance each other. These forces are:

(a) The weight of the structure;

(b) The loads it carries;

(c) The upward supporting forces, reactions or resistances under or around foundations.

(2) The forces acting upon each piece of the structure must balance each other. These forces are, for each piece:

(a) The weight of the piece;

(b) The loads it carries;

(c) The resistances or reactions at its joints.

(3) The forces acting upon each of the parts into which any piece may be supposed to be divided must balance each other.

The **Stability of a Structure** requires the fulfilment of conditions (1) and (2), that is, the ability of the structure to resist the DISPLACEMENT of any of its parts.

The **Strength of a Piece or Member** consists in the fulfilment of condition (3), that is, the ability of a piece to resist BREAKING.

The **Stiffness of a Piece or Member** consists in the ability of a piece to resist BENDING.

The **Theory of Structures** is divided into two parts:

(1) That which treats of strength and stiffness, dealing only with single pieces and generally known as the STRENGTH OF MATERIALS or the MECHANICS OF MATERIALS, before defined.

(2) That which treats of stability, dealing with the structures themselves.

Stress is an internal force that resists a change in shape or size caused by external forces. When the applied external forces reach certain intensities internal stresses hold them in equilibrium.

The **Intensity of a Stress** is measured by the UNIT STRESS. (See Unit Stress.) The INTENSITY OF THE STRESS per square inch on any normal surface of a solid is the total stress divided by the area of the section in square inches. Thus, if a bar 10 ft long and 2 in square has a load of 8 000 lb pulling in the direction of its length, the stress on any normal section of the bar is 8 000 lb; and the intensity of the stress per square inch is 8 000 lb/4 sq in = 2 000 lb per sq in.

Deformation.* When a solid body is acted upon by an external force an alteration takes place in the volume and shape of the body, and this alteration is called the DEFORMATION. In the case of the bar given above, the deformation is the amount that the bar stretches under its load.

The **Ultimate Strength** is the highest unit stress a piece of material can sustain and it is the unit stress at or just before rupture.

The **Working Unit Stress** is the ultimate stress divided by the factor of safety.

The **Safe Load** is the load that a piece can support without exceeding the working unit stresses.

* In mechanics the term STRAIN is now synonymous with the term DEFORMATION. On account of the tendency to confuse the terms STRAIN and STRESS the term DEFORMATION is used to denote change in shape and the term STRAIN is omitted in all discussions in the Pocket-Book.

The Factor of Safety * of a piece of material under stress is the r the ultimate strength of the material to the actual unit stress on the s area; or it is the number by which the ultimate unit stress must be divi give the working unit stress. In designing a piece of material to sustair tain load, it is required that it shall be perfectly safe under all circumst and hence it is necessary to make an allowance for any defects in the ma workmanship, etc. It is obvious, that, for materials of different compo different factors of safety are required. Thus, steel being more homog than wood and less liable to defects, does not require as high a factor of Again, different kinds of stresses require different factors of safety. T long wooden column or strut requires a higher factor of safety than a w beam. As the factors of safety thus vary for different kinds of stress materials, the proper factors for the different kinds of stresses and conditio given in considering the resistance of the various materials to those s under varying conditions.

The Unit Stress is the stress on a unit of section-area, and is usually exp in pounds per square inch. (See Intensity of Stress.)

Dead Loads and Live Loads. The term DEAD LOAD means a load applied and increased gradually and that finally remains constant, such weight of a structure itself.

The term LIVE LOAD means a load that is applied suddenly and causes tions, such as a train traveling over a railway bridge. It has been fou experience that the effect of a live load on a beam or other piece of m has twice the destructive tendency of a dead load of the same magnit intensity. Hence a piece of material designed to carry a live load shoul a factor of safety twice as large as one designed to carry a dead load load due to a crowd of people walking on a floor is usually considered to p an effect which is a mean between that of a dead load and a live load, suitable factor of safety is adopted accordingly. In municipal ordinanc laws relating to the allowable loads for floors, the loads to be supported floors, exclusive of their inherent construction and stationary fixtures, ar erally referred to as the LIVE LOADS no matter of what they may consis the term does not have the exact significance given to it by many en and as explained in the paragraph above.

The Modulus of Rupture or Computed Flexural Strength is the of the UNIT FIBER-STRESS S, computed from the flexure-formula M = when a beam is ruptured under a transverse load. Its value is interm between the ultimate tensile and compressive strengths of a material.

The Elastic Limit is that unit stress at which the deformation of a p material begins to increase in a faster ratio than the applied loads. sometimes called the ELASTIC STRENGTH.

The Modulus of Elasticity or Coefficient of Elasticity. In tr on physics this is often called YOUNG'S MODULUS. It we take a bar of any material, say one inch square, of any length, and secured at one end, and other apply a force, say a certain number of pounds P, pulling in the di

* The ELASTIC LIMITS of materials must be considered in deciding upon worki stresses and in forming a judgment of the security of materials under stress. Wh elastic limit is considered the actual allowable unit stress is made a certain percen it, as 35 or 50%, according to varying conditions. Both ULTIMATE STRENGTHS an TIC LIMITS must be taken into account in practice. But the use of the FACTOR OF S as determined by the old method, is still a great help in the study and applica the principles of the mechanics of materials, and is used frequently in the Pocke

length, we shall find by careful measurement that the bar has been stretched ...ted by the action of the force. If we divide the TOTAL ELONGATION e, ...es, by the original length l of the bar, in inches, we shall have e/l, the ...NGATION ϵ, or the elongation of the bar per unit of length; and if we ... the unit stress S, developed (that is, in this case, the external force P, ...d by the area of the cross-section A, or P/A) by this ratio we shall have ...is known as the MODULUS OF ELASTICITY, E. Expressed in symbols and ...ations, $E = S/\epsilon = \dfrac{P/A}{e/l}$. Hence, we may define the MODULUS OF ELAS... ...as the ratio of the unit stress to the unit deformation. Another definition ... force which would elongate a bar of 1 sq in in cross-section to double its ...l length, if that could be done without exceeding the ELASTIC LIMIT ... material. This is evident from the above equation; for if $A = 1$ and ... E will equal P. These formulas apply only when the unit stress S or P/A ... than the ELASTIC LIMIT of the material. ϵ is an ABSTRACT NUMBER, ...e e and l are both linear quantities, and hence E is expressed in the same ...as S, that is, in POUNDS PER SQUARE INCH.

...an example of one method of determining the modulus of elasticity of any ...ial the following illustration is given:

...pose we have a bar of wrought iron, 2 in square and 10 ft long, securely ...ed at one end, and to the other end we apply a tensile force of 40 000 lb. ... force causes the bar to stretch, and by careful measurement we find the ...tion to be 0.0414 in. As the bar is 10 ft, or 120 in long, if we divide ...4 by 120, we shall have the elongation of the bar per unit of length. Per...ing this operation, we have as the result, 0.00034 in. As the bar is 2 in ..., the area of cross-section is 4 sq in, and hence the stress per square inch ...000 lb. Dividing 10 000 by 0.00034, we have, as the MODULUS OF ELAS...ITY of the bar, 29 400 000 lb per sq in. This is the method generally employed ...ermine the MODULUS OF ELASTICITY of iron ties; but E can also be deter...d from the DEFLECTION of beams, and it is in that way that its values for ... woods have been found. The modulus of elasticity is used in the deter...tion of the STIFFNESS of beams.

...e **Moment of a Force** with respect to an axis is the product obtained ...ltiplying the magnitude of the force by the shortest distance from the ... to its line of action. The shortest distance is called the LEVER-ARM of ...force. The moment of the force is the measure of the tendency of the force ...se ROTATION about the axis. (See Chapter VI and IX.)

...e **Center of Gravity** of a body is the point in the body through which ...ESULTANT of the forces exerted by gravity upon all the particles of the ... passes. A body may be balanced upon a point placed above or below ...center of gravity, because the RESULTANT of any number of forces may be ... in equilibrium by an equal and opposite force. Another definition of the ...ER OF GRAVITY of a body or bodies is: a point such that there is NO TEND...Y TO ROTATION about any axis drawn through it. (For center of gravity ...faces, lines and solids, see Chapter VI.)

2. Classification of the Principal Stresses Caused in Bodies by External Forces

...nsion is the stress that resists the tendency of two forces acting away from ...other to PULL APART two adjoining planes of a body.

...pression is the stress that resists the tendency of two forces acting toward ...ther to PUSH TOGETHER two adjoining planes of a body.

Shear is the stress that resists the tendency of two equal parallel f ing in opposite directions to cause two adjoining planes of a body to on the other.

Torsion is the stress that resists the tendency of forces to TWIST a

Combined Stresses. Parts of structures are often acted upon b external forces which develop stresses of different character, such as flexure and compression, flexure and tension, flexure and torsion, shear compression or tension, torsion and compression, etc.

CHAPTER II

FOUNDATIONS

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1. Definition of the Word and Terms Used

finitions. The word FOUNDATION is derived from the Latin verb FUNDARE ing to establish or lay the base, bottom, keel or foundation of anything. English word is used in the broadest possible way to describe the base, ial or otherwise, on which anything is supported, and in technical language y be used to describe any part of a structure on which a subsequent oper- or construction is superimposed. Thus a plaster wall may be called the ATION for a fabric to be stretched thereon and the fabric in turn becomes OUNDATION for various coats of paint or other decorations. More specifi- ed in relation to a building or other complete structure the word FOUNDA- is unfortunately applied indiscriminately (1) to construction below grade, as footing courses, cellar walls, etc., forming the lower section of the struc- (2) to the natural material, the particular part of the earth's surface on h the construction rests; and (3) to special construction such as piling or used to transmit the loads of the building to firm substrata. In view of definite meaning of the word it is advisable to use it either to distinguish below grade, or below the tier of beams nearest to grade, from work above . In even a still more restricted sense, it might include only the work the cellar or basement-floor to rock or other solid foundation-bed. Chapter II, Subdivision 29, Chapter III, Subdivision 2, and Water- ing for Foundations, Part III.)

e Foundation-Bed. The natural material on which the construction s called the FOUNDATION-BED. Walls, piers and columns below grade lled, in general, FOUNDATION WALLS, PIERS AND COLUMNS to distinguish from similar construction above grade and occasionally those only the basement-floor are so called; the lower portions of walls, piers or ns which are spread to provide a safe base will be called FOOTING COURSES.

2. General Requirements

e Object of Foundations. The object to be borne in mind in designing undation is to provide a safe and permanent base for the superstructure that the movement of the base and of the superimposed structure shall be st possible and shall result in the least possible damage to the structure. ly meet the above requirements the design and construction should ful- e following conditions:

The Materials of Construction should be proof against all deteriorating ces, or, if any of the materials are liable to deterioration they should be ently protected.

Stresses and Future Changes. No part of the foundation-structure l, under any combination of loadings, be stressed beyond safe limits, and ibility of future additions or changes in the superstructure, or of a change use of the building, should be kept in mind.

(3) **The Load on the Natural Bed** should be kept within the safe limit f material, under the worst conditions to which it may be exposed. I this limit the amount of settlement allowable will in many cases detern limit rather than the safe ultimate bearing capacity.

(4) **Adjoining Excavations.** The possible danger to the structure o stability of the foundation-bed from adjoining excavations or other di causes should be guarded against as far as possible.

Physical Conditions of the Site. In order to meet the above requir the design should be suited to the physical conditions existing at the l The architect or engineer should personally examine the site. He shoul all available information relative thereto and, if necessary, should make and tests so as to secure reliable information on which to base his design foundation. The first step is, therefore, a detailed and exhaustive stud site to determine the characteristics of the foundation-bed on which th ture is to rest.

3. Geological Considerations

Character of the Foundation-Bed. A knowledge of geology is of assistance in many cases in making a proper estimate of the characte foundation-bed. While it is not proposed in the limits of this chapter to any general geological discussion the following notes may be of value in the architect to determine whether any given deposits can be relied affording a stable foundation-bed. Broadly speaking, as the location building may be in any part of the world, so the materials encounter belong to any one of the many geological formations forming the surfac earth. For practical purposes, however, the materials met with are divided into ROCK, or materials other than rock, roughly defined as EA

4. Composition and Classification of Rocks

Composition of Rocks. Rocks, and the earthy deposits derived fro are composed of various minerals of which many hundred kinds are each varying from the others in some particular of chemical compositio of crystallization or other characteristic. A rock or an earthy deposit m sist almost entirely of a single mineral, but it is usually composed of distinct minerals or of mixtures of minerals. The principal classes c forming minerals are:

(1) **The Silica Minerals**, composed of silica (SiO_2) in different forms;

(2) **Silicates** or combinations of silica, with various metallic bases;

(3) **Calcareous Minerals** composed of calcite or carbonate of lime (and its combinations.

(1) **Silica Minerals** are different forms of the oxide of silicon, kn SILICA. In the crystalline state silica is known as

Quartz. This is the most abundant of all minerals. Owing to its h and insolubility it resists decomposition and abrasion better than the with which it is associated and grains of it form the principal constit sandy deposits. In finely comminuted particles it forms a part of mos clays.

Flint, Chert, Agate, etc., are non-crystalline varieties of silica. SIL forms the cementing material in many sandstones and other rocks.

(2) **Silicates** or combinations of silica with various bases are second portance only to quartz.

...r, an important constituent of granite and other igneous rocks, is a ...of alumina with potash, soda or lime. When exposed to the action of ...it slowly decomposes, forming silicate of alumina, the base of clay. The ...position of granite results in the formation of clay and crystals of quartz ...ica. The mica is very slowly affected and the quartz is practically un...ed.

...a. The various mica minerals are silicates of alumina, with potash and ...constituents. All varieties are soft and split into thin elastic plates. ...particles of mica are frequently found in sand.

...blende and Augite are silicates of lime, magnesia, iron and alumina and ...frequent occurrence.

...ite, Talc and Soapstone Travertine are hydrated silicates formed from ...silicates by a chemical change in which a certain amount of water is ...bed. These minerals are soft and have a SOAPY FEEL. Special care should ...ken in building foundations on rock of this character to guard against any ...g on the foundation-bed or between parts of the foundation-bed.

... Calcareous Minerals. The following are the principal calcareous miner...

...ite ($CaCO_3$), carbonate of lime, when pure and crystallized, is known as ...ND SPAR. It is soluble in water containing CO_2. Calcite in varying de... of purity forms limestone and marbles. As a result of its solubility ...es and voids are frequently found in limestone.

...mite is a carbonate of lime and magnesia. It forms the so-called DOLO... LIMESTONES, which are less soluble than the calcite limestone.

...ite, Gypsum, Alabaster, Anhydrite, Aragonite and Apatite are other and ...portant lime-minerals.

...sification of Rocks. Rocks are classified not only according to the ...als of which they are composed, but also according to the way in which ...have been formed, as:

Igneous Rocks, which have solidified from a molten condition;

Sedimentary Rocks, which have been formed under water by mechani...ssure or by cementation due to chemical or organic proceesses;

Metamorphic or Plutonic Rocks, which have changed from their original ...cter as igneous or sedimentary rocks.

Igneous or Plutonic Rocks are not truly stratified. They may be ...lar, crystalline or glassy in texture. GRANITE, SYENITE, BASALT, TRAP, etc., ...amples. LAVA, PUMICE and OBSIDIAN are volcanic products, as are also ...e deposits of mud and ash. With the exception of volcanic ash and mud, ...neous rocks are enduring and are not liable to present any unforeseen weak...s in foundation-beds.

Sedimentary Rocks are composed of sand, clay and other materials ...ng from the breaking down of the original igneous rocks. These materials ...eposited in horizontal beds generally by settling from water, and the con...ion into rock was generally affected under water by chemical, mechanical ...nic action. The resultant rock-masses are stratified as a result of their ...ent materials having been deposited in layers. As sand and clay are ...st abundant products of rock-decomposition, so the sedimentary rocks ...st frequently SILICEOUS (sandy) or ARGILLACEOUS (clayey).

...stone is composed of grains of sand cemented together by silica, oxides ... or carbonate of lime. The durability of sandstone depends on the solu...

bility of the cementing material. Carbonate of lime being soluble, sand containing it as cementing material yield to the weather and are not so as sandstones having silica or iron oxide as cementing material.

Argillaceous Rocks contain clay with fine sand, mud, etc., and while SHA some other varieties are compact and hard when first uncovered, they ar to deterioration when exposed to frost, water and other disintegrating ag

Limestone is composed more or less of carbonate of lime derived fr calcareous skeletons of marine animal and vegetable organisms. The acter of limestone varies greatly. In so-called FOSSILIFEROUS LIMES fossils of shells or corals indicate clearly its origin, but in other limestone are no fossils or other indications of the organic origin of the calcareous m Admixtures of sand, clay, or other impurities may make it difficult to dist certain limestones from sandstones or shales.

Dolomite is a limestone containing a high percentage of magnesia.

Hydraulic Limestone is a limestone containing clay.

Chalk is a soft limestone composed of the fine shells of minute marine isms. In general, the purer the limestone the more soluble it is and the the danger from fissures or caverns due to the action of water.

(3) **Metamorphic or Plutonic Rocks** are rocks which have been from sedimentary or igneous rocks by heat, compression, or moisture, alone or in combination. Thus by heat from a nearby intrusion of molte limestone is changed into a crystalline marble. The general effect of MORPHISM is to produce a hard or durable rock.

Quartzite, a metamorphosed sandstone, is a crystalline rock of great h and durability.

Slate is a hard dense rock, sometimes with a well-defined tendency to s thin plates. It has been formed by metamorphic action from clayey sha is generally durable, but liable to slide along planes which are sometim allel to the cleavage, or along seams which are not parallel to the cleavag

Gneiss is a "laminated metamorphic rock that usually corresponds alogically to some one of the plutonic types." * There are many varieti classified in accordance with the igneous rocks to which they most nearly spond in composition. Some varieties resemble granite, but the lamin striped aspect is generally characteristic. They are generally compa durable.

Schists are similar to gneiss but are more finely foliated or striped. I SCHIST there are layers or foliations composed of fine grains or plates c Mica-schists are liable to decomposition and it frequently happens that tions have to be carried to great depths through decomposed rock of th acter before solid rock is encountered. The material resulting from the position of this rock contains fine grains of mica and other fine mater when wet, acts as QUICKSAND.

Rock as a Foundation. All rock, if sound and not liable to slip proverbially a solid foundation and capable of supporting any weight building is likely to impose on it. Care should be taken that rock l disintegration is protected from the weather, water-action, or other disi ing influences.

5. Geology of Earthy Material

Earth and Soil. Materials other than rock, resulting from the dis tion of rock-masses, are broadly classed as EARTH. The word SOIL, wh

* Kemp.

...gnate any earthy material not rock, is a misnomer, in that the idea of ...ty, or the lack of it, is conveyed when the word SOIL is used.

...he agencies producing the disintegration of the rock masses which form or ...ke the entire surface of the earth, are various, but for the purpose of this ...er they may be defined as (1) CHEMICAL and (2) MECHANICAL.

...Chemical Agencies. By CHEMICAL ACTION OR DECOMPOSITION, a rock ...of great strength and hardness and of complicated mineralogical structure ...disintegrate into a noncoherent mass of elementary minerals. Thus a ...thic granite under the combined action of water and varying temperature ...egrates, the crystals of feldspar changing chemically and forming the ...ted silicate of aluminum known as CLAY, while the crystals of quartz, ...or hornblende, being more resistant to chemical action, retain their chem... ...identity but become detached particles of SAND.

...Mechanical Agencies. By the MECHANICAL AGENCIES, such as the action ...st, moving water or ice, fragments of rock are detached from the ledge of ...h they originally formed part and are subsequently transported, by the ... of glaciers or streams, or by the wave-action in bodies of water. The ...ion between the materials thus roughly thrown about breaks up the rock-...es into smaller and smaller pieces without altering the composition of the ...material.

...ving Water. As flowing water more readily transports small particles ...large ones, the larger pieces of rock move intermittently during periods of ...or flood and are deposited as soon as the velocity of the water falls; while ...maller particles are held in suspension longer and, as the velocity of the ... falls, are deposited in the order of their size, the largest first. The ...upper courses of streams and rivers in mountainous regions constantly roll ...ind together the materials in their rocky beds, the heavy masses being ... slowly. The attrition between the fragments forms GRAVEL and SAND ... are washed down stream to be deposited, as the current slackens, first as ... of GRAVEL, then as SAND-BARS, and finally, in the slow-moving lower ... as BEDS OF SILT and ALLUVIUM.

...ciers and Glacial Deposits. The action of glaciers is similar to the ac-... of streams. Glacial deposits, the so-called GLACIAL DRIFTS, are composed ...d clay, gravel and boulders but, in general, there is a noticeable differ-... between glacial deposits and deposits made by rivers or streams. In glacial ...its the boulders frequently exhibit groovings or scratches on their faces ... the edges and surfaces of the boulders are generally sharp, so that a ...er may appear as if it had been recently fractured. They rarely exhibit ...ooth, water-worn and rounded surfaces found on boulders formed by ...action. Moreover, the glacial boulders may be found singly, or unas-...ed with other boulders in a deposit of sand or gravel. The deposit differs ... river-deposit in that there is no classification as to size; the boulders may ... on the surface or may be disseminated as if by accident through the sand ...avel forming the body of the deposit. Such glacial deposits partake of ...racter of a rough artificial fill without the stratification or classification ...ze which is characteristic of river-deposits. In glacial moraines or dump-...ounds it not infrequently happens that the surface-water finds underground ...s forming so-called SINK-HOLES. A line of glacial deposits extends across ...tinent of North America from Long Island westward. The southern ...an be determined by reference to geological maps.

...ial and River-Deposits Distinguished. It is important to distinguish ... GLACIAL and RIVER-DEPOSITS, because, while the occurrence of glacial ...s gives, in general, little or no information as to the character and value

of the surrounding deposits, the occurrence of boulders, on the other h river-deposits is generally an indication that the bed of which they form has been thoroughly consolidated as a result of the river-action which fon and, also, because such deposits generally extend down to rock or to som pact material which at the time the deposit was made was capable of r the action of rapidly flowing water.

Wave-Action on Lakes and Along Coast-Lines is constantly worl the materials composing the beach. Rock-masses are broken away fro and ground together, producing boulders, gravel and sand. The sand carried more readily by the tidal currents, is deposited in the more shelter tions and forms BEACHES, while the larger rock-masses remain near th of origin in BARS and REEFS.

Beds of Sand, Gravel and Boulders deposited by the action of w the SHORES OF SEAS OR LAKES are not necessarily constant in character a should be made to determine the character of the material underlyi BEACH-FORMATIONS. In large river-valleys where the general forma composed of silt or other fine material little reliance should be placed occurrence of BEDS OF GRAVEL, even if such beds extend over large areas. should be made to determine that such beds are not underlain by les worthy materials. Where tributary streams discharge into large valle may deposit BARS OF SAND, GRAVEL and BOULDERS on top of the silt, other materials formerly deposited by the main river. (See page 136 general topographical conditions should serve as an indication of danger cases.

Results of Chemical and Mechanical Action. As a result of t going brief description of the agencies at work it may be seen that IC and STREAM-ACTION alike tend to disrupt rock-masses and to produce b gravel, sand and finer materials. The ultimate result of the combin CHEMICAL ACTION and MECHANICAL ACTION is to reduce the hardest rock finest sand, the most impalpable clays, silts and muds; and the ACTION C WAVE and MOVING WATER is to classify such materials in deposits of g uniform size.

6. Materials Composing Foundation-Beds

Classification and Definitions. The following list includes the r which are most frequently encountered, with their definitions.

Rock (solid rock, bed-rock, or ledge). Undisturbed rock-masses for undisturbed part of the original rock-formation.

Decayed Rock (rotten rock). Sand, clays and other materials from the disintegration of rock-masses, lacking the coherent quali occupying the space formerly occupied by the original rock.

Loose Rock. Rock-masses detached from the ledge of which they o formed a part.

Boulders. Detached rock-masses larger than gravel, generally roun worn as a result of having been transported by water or ice a consider tance from the ledges of which they originally formed a part.

Gravel. Detached rock-particles, generally water-worn, rounded a mediate in size between sand-particles and boulders.

Sand. Non-coherent rock-particles smaller than ¼ in in maximum sion.

The material resulting from the decomposition and hydration of feld-rocks, being hydrated silicate of alumina, generally mixed with powdered quartz and other materals.

Hard-Pan. Any strongly coherent mixture of clay or other cementing with sand, gravel, or boulders.

A finely divided earthy material deposited from running water.

Finely divided earthy material generally containing vegetable matter deposited from still or slowly moving water.

Loosely used to describe any earthy material.

Earthy material capable of supporting vegetable life and generally to material containing decayed vegetable or animal matter.

Earthy material containing a large proportion of humus or vegetable

Earthy material containing a proportion of vegetable matter.

Compressed and partially carbonized vegetable matter.

7. Characteristics of the Materials of Foundation-Beds

Rock, or, as it is locally known, BED-ROCK, or LEDGE, is proverbially foundation. The harder rocks, such as granite, trap, slate, sandstone, etc., are all capable of carrying the load of any ordinary structure. rocks, among which may be classed the shales, shaley slates and marley limestones and clay stones, should not be loaded with more than per sq ft unless they are tested for greater loads. In all cases where are to be placed on what is supposed to be solid rock, care should be to determine whether or not the supposed solid consists of a detached and, also, in case the bedding-planes of the rock are inclined, if there is from a slip of the layer forming the foundation-bed. (See pages 139 and to side-slope locations.)

Decayed Rock. Certain igneous or metamorphic rocks such as granites, etc., frequently disintegrate, forming so-called ROTTEN ROCK or DECAYED The decayed rock is generally found in conformity with the ledge of it originally formed a part. It may retain the stratification, color and of the solid rock, but as a result of the disintegrating effect of water agents, it has lost the solid character of the original rock. When with a hammer it does not give the characteristic ringing sound of solid It may be fairly compact and hard, or so soft as to be readily excavated pick and shovel. The amount of such disintegrated rock overlying the rock varies greatly; in some cases the removal of a few inches will disclose rock, in other cases the layer of decayed rock may be many feet in thick-Test-borings in rotten rock give samples similar to the samples from solid so that it frequently happens that while the foundations are planned for the excavations disclose a thick layer of rotten rock. In such cases, impracticable to carry the footings down to solid rock, it may be necessary the size of the footings or to adopt some other expedient.

Rock. Where a rock-mass detached from the ledge of which it orig-formed a part is encountered it must not be loaded in excess of the safe of the material by which it is surrounded. If the voids between ad-pieces of loose rock are completely filled in with hard-pan, compact or clay, the loading may be the same as for the filling-in material. should be taken to determine that no voids exist. In natural rock-

fills, as in artificial rock-fills, it may happen that large voids exist betw rock-masses, forming passageways for streams of water, in which case t extreme danger of settlements.

Boulders, Gravel and Sand. Boulders are rock-masses which hav transported by water or ice-action. Boulders are sometimes found di ated through sand and clay and in such cases the load should be limited safe load of the material in which they are found. At other times bould found in beds, packed closely together, with the interstices filled in with sand, or clay. In such cases it is usually safe to assume that no furth solidation of the mass is likely to take place. If the bed of boulders ext rock, they will safely sustain any load which will not crush them.

Gravel. The name GRAVEL is given to rock-particles larger than sa smaller than the rock-masses known as BOULDERS. If compact, an underlying bed of poorer material exists, gravel forms a most desirable tion-bed, equal to sand or boulders in supporting power and not as l be disturbed by adjoining excavations or pumping operations. If ce it may partake of the quality of hard-pan or rock. Care, however, sh taken to determine whether or not the bed of gravel has been deposited layer of silt or quicksand. It is possible for this dangerous condition t (See page 134.)

Sand. Sand is composed of comminuted rock-material. As quart most abundant rock-mineral and as its hardness and insolubility make i resistant to disintegrating action, it will be found to be the principal con of most deposits of sand or sandy material. Grains of mica, feldspar and other minerals are frequently found. Sand is described as bein MEDIUM, or COARSE, according to the size of the grains of which it is co

Coarse Sand may contain particles of gravel, but after elimina particles which will not pass a screen with 4 meshes to the inch it will l that a large proportion of the remaining material is too coarse to pass a sieve.

Fine Sand, on the other hand, may contain no particles which will a 20-mesh sieve, and a considerable proportion which will pass a 100-me

Very Fine Sand is frequently mistaken for clay and, indeed, genera contain some clay, as clay generally contains fine sand.

Uniform Sand is sand in which there is relatively a small variatio size of the particles.

Balanced Sand is sand in which the size of the particles varies from small and in which there is no great difference in the numbers of pa each size.

Clean Sand contains no clay or loam, but a pure sand containing a l centage of fine particles is often considered to be NOT CLEAN.

Sharp Sand is clean sand containing coarse, angular grains. Whe grasped in the hand it gives a NOTE, due to the particles slipping over ea Sharp sand is generally esteemed for use in mortar, although it requi cement to fill the voids and, in the writer's opinion, is not as desirable as rounded sand.

Rounded or Buckshot Sand is composed of rounded grains not c together.

Quicksand. This term is popularly used to describe any fine sand, ture of fine sand and clay, which, when wet, forms a soft, unstable

popular mind quicksand is supposed to have some mysterious and peculiar properties which result in a tendency to FLOW LIKE WATER and to SUCK IN animate and inanimate objects. These manifestations are connected with various theories as to the composition of quicksand, some persons insisting that quicksand must contain flakes of mica or some slippery mineral, others that the particles must be extremely fine or spherical in shape, while others contend that there must be a certain proportion of fine clay with the sand. The fact is that any uncemented sand, when subjected to the action of moving water, will move and that any sand moving as the result of the action of water becomes quicksand. The finer the sand the more readily it is affected by a current of water, so that fine sands are more troublesome than coarse sands. A coarse sand having large voids, permits the flow of a certain amount of water through it; if this flow has not sufficient velocity to disturb the particles of the sand, the sand can be drained without moving it. In a fine sand, having very small voids, a similar flow of water will cause the whole mass to move and there is great difficulty in draining it without producing a current sufficient to cause it to move or flow.

Excavations in Quicksand are made difficult by the tendency of the sand forming the sides of the excavation to flow into the excavation; and even if the sides of the excavation are protected, it not infrequently happens that the bottom of the excavation will LIFT, that is, there will be a movement of material from points outside of the line into the excavation, the movement in general following a curved line, and carrying the sand, under the protected side walls of the excavation. In such cases some advantage may be gained by surrounding the excavation with driven wells and draining the soil by continued pumping through sand; in other cases, wooden or steel sheeting may be driven to a point below the depth to which the excavation is to be carried, or to some underlying layer of impervious material, in which case the sheeting will act as a coffer-dam to cut off the flow of material. Such sheeting, however, must be practically water-tight, as extremely fine sand, when in the condition of quicksand, will flow through very small apertures.

Quicksand as a Foundation-Bed is objectionable on account of the danger of its moving or flowing, in case it finds any outlet such as would be afforded by an adjoining excavation. Cases are known where excavations have permitted the escape of quicksand and resulted in the settlement of buildings at a very considerable distance. Such settlements have occurred not only when the footings themselves rested on quicksand, but also when they were on a stratum of coarse sand, gravel or clay of good quality which rested on an underlying stratum of quicksand.

Pockets of Quicksand. It frequently happens that pockets of fine sand are found in deposits of mixed character. Where such pockets are small in extent the fine sand may be removed and the spaces filled with concrete. Where the pockets are larger it may be necessary to carry piers through them to a better foundation-bed, to drive piles, or to resort to other expedients.

Fine Dry Sand is readily converted into quicksand by the addition of water, and this fact should be carefully borne in mind in considering the load on fine sand, as a material which in dry weather is apparently safe, may be, in wet weather, an extremely dangerous one. It is frequently stated that confined quicksand is a perfectly reliable material on which to found a building. While this, as a theory, cannot be controverted, it is a dangerous assumption to act on because of the impossibility of providing that the fine sand shall be always confined.

Variation in the Size of Grains of Sand. The accompanying diagram (Fig. 1) shows graphically the results of sieve-tests on characteristic sands.

The dash-line curve (1) is an average, giving the results of sieve-tests on so-called quicksands; the full-line curve (2) gives the result of sieve-tes natural sand which would be classed as a good building sand; the dot-an curve (3) gives the result of sieve-tests on a fine beach sand remarkable

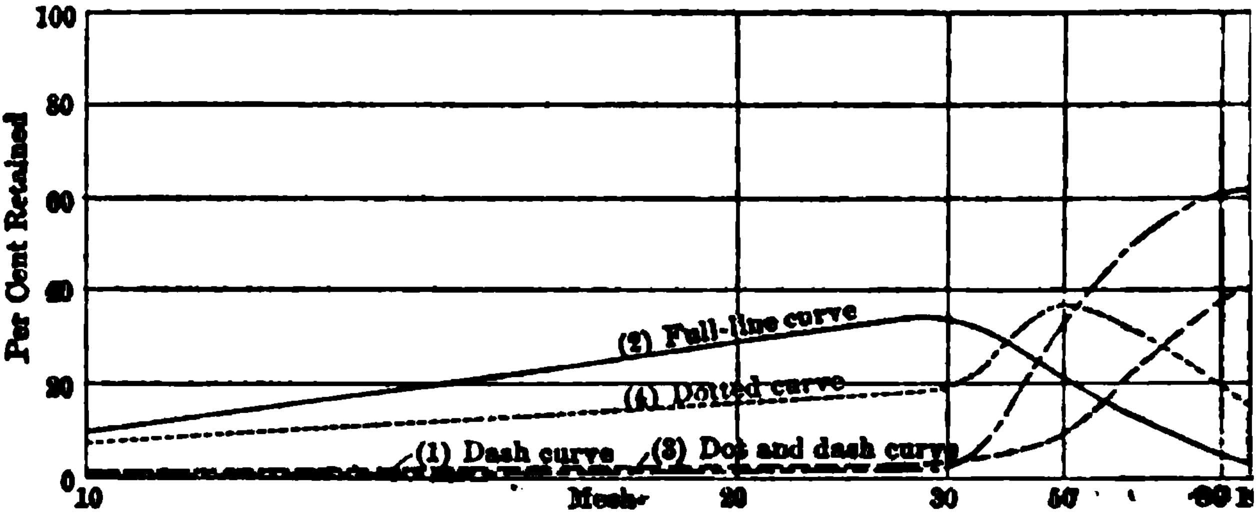


Fig. 1. Graphical Illustration of Results of Sieve-tests on Sands

uniformity of the size of its grains. For purposes of comparison and i to show the variation in sands which appear to be substantially the sa dotted curve (4) has been added. This shows the result of tests on a ban apparently as coarse as sand (2), but containing a much larger percent fine particles between 0.015 and 0.0055 in in diameter. Fine sand freq contains a considerable proportion of clay. A chemical analysis of a so QUICKSAND from the down-town section of New York City, reported on writer by Dr. C. F. McKenna, is as follows:

Mark: "Commercial Cable"

Silica	73.76
Alumina and oxide of iron	18.52
Lime	1.60
Magnesia	1.48
Loss on ignition	2.26

A rational analysis shows the following composition:

Quartz, as given	39.38
Clay and mica, as given	23.94
Feldspathic detritus	36.68

On the other hand, a sample of extremely fine sand from Michigan, of 75% passed a 200-mesh sieve, appears to be absolutely pure quartz.

Clay. When pure, clay consists of hydrated silica of alumina, the p of decomposition of feldspar. Ordinarily, various impurities are mixed w clay, so that, in general, clay may be considered a mixture of hydrated of alumina with other finely divided minerals. Mixtures of clay and sa found, varying from beds of nearly pure clay to beds of nearly pure san no definite classification can be made.

The Effect of Moisture on Clay. Clay as generally found in excav is in a plastic condition due to the presence of moisture, the amount of present varying greatly. On drying, the clay shrinks in volume and lo plasticity, becoming a firm and coherent mass resembling in consistency dried brick. Large masses of clay are liable to crack into a number of

...ents during the process of drying, as the result of the shrinkage in volume. ...n these lumps are crushed or ground the clay becomes an extremely fine ...palpable powder. The loss in volume due to the change in the condition ...the clay from a moist, plastic state to a thoroughly air-dried condition may ...ount to from 10% to 20% of the original volume. Compact, moist clay is ...rvious to water in the sense that water cannot pass through it as it would ...ugh porous sand; but when clay is exposed to water the clay gradually ...rbs the water, so that eventually the entire mass becomes saturated and ...ened by the water.

...lay as a Foundation-Bed. Clay is not a reliable material on which to ...d a building; first, because of the PLASTICITY of the clay when wet, and ...dly, because of its TENDENCY TO SHRINK on losing its contained moisture. ...e plasticity of clay increases with the percentage of contained water, so that ...n, hard clay may be converted into a liquid puddle by being agitated in the ...ence of a sufficient amount of water. The plasticity is also increased by ...sure, as is shown by the action of clay in a brick-machine. Clay, in a founda...-bed under moderate pressure imposed on it by the footings of a structure, ...uently develops this QUALITY OF PLASTICITY, the clay moving out from be...th the footing and causing serious settlements and displacements of the ...ings. This movement of the clay may be a local movement, as referred to ...h footing, in which case the clay flows from beneath the footing laterally ...ard the side and then upward, causing the surface of the adjacent ma...al to rise and to form so-called BULGES or WAVES. If this motion is uniform ...m the center toward the sides, the footing may settle vertically, but more ...erally the movement will not be symmetrical and the footing will settle ...re on one side than on the other. Such movements of the clay may be re...ed or prevented in some cases by the simple device of loading the surround... soil, as, for example, by a concrete floor.

...ovements of Clay Foundation-Beds. The movement of the clay may ...e a larger scale, amounting to a general flow of the clay underlying the ...ire building toward some point where the pressure on the clay is less than ...e pressure resulting from the load of the building. Such general movements ...e more likely to happen if the building is located on the side of a hill, so that ...e clay finds some outlet at a point below the level of the footings. It fre...ently happens that adjoining excavations cause settlements to buildings at a ...siderable distance, by affording an outlet to a bed of clay. As noted else...ere (pages 135 and 146), beds of clay resting on inclined strata of rock or ...er material are liable to move downward, sometimes with a slow, almost ...perceptible movement, and at other times forming landslides of greater or ...s magnitude.

...otection of Clay Foundation-Beds. Where the foundation-bed is clay ...ed with a considerable amount of clay, it is advisable to protect it from ...er-action, so far as is possible, by a system of drains surrounding the site ...the building and by diverting the surface-water from the building. Care ...ld be taken in back-filling around exterior walls to prevent any accumula...n of water which might affect the material under the footing. The neglect ...such precaution has frequently resulted in serious settlements during, or ...ediately after, construction.

...d, Silt, Peat and Other Unstable Materials. When the site of a ...ture is in a marsh or on materials which are not capable of affording a ...foundation, the only alternative is to resort to the use of wooden piles, ...ete piles, or piers sunk to an underlying and firmer strata. Such special

constructions will be described under Subdivisions 27, 28 and 29, which cc wooden piles, concrete piles and piers sunk by the coffer-dam or c methods.

Filled Ground. All artificial fills and some natural fills are liable to a or less uniform but continuous settlement or shrinkage due to the gradua solidation of the material of which the fill is composed. Where the fill is o rock this consolidation may amount to little, but where the fill is of eartl especially where it is of mixed materials, the shrinkage will not only be la amount but will continue for a very long period. For example, wher has been thrown on top of a rock-fill each rain-storm will wash some of th into the voids in the rock-fill, and this action will be continuous until the voids are filled in. Any vegetable matter, or other matter liable to and shrinkage in volume, will increase the total shrinkage of the mass. tain natural deposits, such as beds of peat or soils containing vegetable m are apt to shrink in volume from the same causes. When it is necessa found a building on such material it is inevitable that the footings will with the mass, notwithstanding that the unit load on the foundation-bed small as to be negligible. In such cases the settlements may be vertica uniform; but if the depth of the fill under one part of the building is gr than the depth under another part, the settlements will not be uniform, a shrinkage in the fill will, in general, be in proportion to the depth of th No important building should be founded on such material and, whe possible, the footings should be carried down through the filled-in mater some more reliable underlying stratum.

8. Allowable Loads on Materials of Foundation-Beds

General Considerations. Owing to the infinite number of variations i materials encountered and the conditions affecting the reliability of such ı rials, no general or definite rule can be given, and every case should be care investigated before determining the allowable unit load on the foundation If the material and conditions are uniform over the entire site of the bui a uniform unit load may be used, but in practice it is frequently found entirely different conditions exist under different portions of the same bui and in such cases great care must be exercised in determining the unit l For instance, one section of a building may rest on rock and another sectic a light compressible soil or on a clay of doubtful stability. In such case unit load on the compressible soil or on the clay must be reduced as muc possible so as to reduce the differences in settlements between the two sec of the building to a minimum. If the entire building were on a compres soil a very considerable settlement might be allowable, provided it was unif but in this particular case it is known beforehand that the part of the buil on rock will not settle at all and that any settlements of other parts of building must be considered as unequal settlements, and, as such, liable to duce cracks and distortions in the building. It is also important to remea that a certain unit load on compressible soil may be safe, in that the soil ultimately safely support that load; but the use of that load would neverth be inadvisable on account of the excessive settlements. In this connectic may be said that a considerable settlement, if uniform, in a detached buil may be a matter of no importance; but that where a building is to be structed in contact with adjoining buildings or where additions are to be n to an existing building, the total amount of settlement becomes a matt prime importance. These and other considerations, such as the characte

posed building and of the material composing it, should be borne in in selecting the unit load for any given foundation-bed, irrespective of the pressure as given by building codes or by examples quoted in this chapter.

Loads on Rock. The safe unit load on rock may often amount to than the crushing strength of brickwork or stone masonry, and in nearly any material worthy of the name of rock is capable of supporting from 30 tons per sq ft.

Loads on Sand, Gravel and Boulders. When compact and con-terally these materials are capable of supporting 10 tons per sq ft with-ppreciable settlement. It rarely happens, however, that it is advisable to such materials with more than 5 tons per square foot.

Loads on Loose Sand. By loose sand is meant sand which has been thoroughly compacted and which may settle by its own weight inde-ntly of a superimposed load. All such materials should be tested and unit load reduced in accordance with the result of such tests.

ds on Fine Sand or Quicksand. It is probable that fine sand, if tely confined, will sustain as heavy a load as coarse sand, but in view of fact that if afforded the slightest opportunity it is liable to lateral displace-t, it is inadvisable to found any structure on such material. When it is ative to place the footings on such material the unit load should be reduced ch as possible and preferably to less than 2 tons per sq ft, and great care ld be taken to connect all footings with a continuous layer of concrete so to prevent any flow of material into the cellar-excavation. Care should taken, also, that any sumps, pump-pits, drainage-arrangements and sewer-ections for the building do not permit the escape of any quicksand..

Loads on Hard-pan and certain cemented sands partaking of the e of hard-pan may approximate rock in hardness and reliability. Such rials, however, are liable to soften if exposed to water. If these materials, uncovered, are dry, experiments should be made to determine how they e when wet, and if the level of the water in the ground is liable to change to reach the layer of hard-pan, the load should be correspondingly reduced. ented hard-pan containing gravel has been frequently loaded with more 10 tons per sq ft. Care should be taken, however, to determine that layer of hard-pan is continuous to a solid substratum, as it frequently happens layers of hard-pan and fine sand or clay are deposited alternately.

fe Loads on Clay. Ordinary clay should not be loaded with more than ns per sq ft. If soft and plastic, a load of 2 tons per sq ft may produce visible settlements. Clay with so large a percentage of sand that it loses plasticity has been loaded with from 4 to 6 tons per sq ft without undue lements, and sand or gravel containing sufficient clay to act as a cementing rial will partake of the qualities of hard-pan. In general, however, clay e most dangerous of all the materials on which structures are founded and unit load should be reduced to a minimum and every precaution taken to ent the flow of material. Undue reliance should not be placed upon load-tests of clayey soils. It is probable that a loading on a large area which produce a movement of the clay will on a small area have no effect, so that unsafe to rely upon the results of a test-load applied to an area smaller than actual supporting areas to be used. From the experience gained in the struction of large buildings in Chicago which were FLOATED on clay, the wable unit load has been generally reduced to 2 tons per sq ft and, in writer's experience, a load of less than 2 tons per sq ft on clay has pro-d settlements varying from nothing to 12 in.

9. Unit Loads on Foundation-Beds Allowed by Building Co

Variations in Building Codes. Table I gives an outline of the requi of different cities as to the allowable unit loads on different mater contained in their respective BUILDING CODES OF REGULATIONS. Wh allowed loads given may in some cases be based upon actual experience respective localities, it is more likely that they are based upon the ind experience of the authors of the codes, or are copied from other codes architect should, therefore, not place too much reliance on the unit loads by the codes, but should investigate each case and determine for him proper allowance to be made.

Special Requirements of Some Building Codes.* The Boston co vides that "the footing shall not overload the material on which it rests

The New Orleans code limits the maximum load to 1 400 lb per sq ft, th city being on an alluvial-delta formation.

The Buffalo code limits the load on SOIL to 3¼ tons per sq ft; if the SOIL than hard clay or gravel the supporting areas "shall be extended as dir

The Cincinnati code limits the load on soils INFERIOR to those listed, t per sq ft.

10. Investigation of the Site

General Considerations. To determine the character of the material will be encountered at the level of a foundation-bed, the architect shou get as definite information as possible from others as to their experience i ing excavations and erecting buildings in that vicinity. In some localit subsoil conditions are uniform over large areas, while in other localities tant variations may occur within the limits of a city lot. Abrupt cha surface-topography, changes in the character of the surface-soil or in the vegetation, proximity to old or existing water-courses are suggestive surface irregularities. In such cases, and in all cases where there is any as to subsurface conditions, a sufficient number of exploratory borings pits should be made to determine the facts. This exploratory work sh below the level of the proposed footings, should determine the ground level and insure that no unsuspected layer of quicksand or other uns material underlies the foundation-bed. The methods in use for such e tions are as follows:

Testing in an Open Pit. For shallow work an open pit is the mo isfactory method as it allows actual inspection of the undisturbed material considerable area. If the excavation is in firm material, no sheet-piling o protection may be required; but if in flowing material, or if carried than adjoining footings, timber sheeting or steel sheeting should be em If the excavation is carried no deeper than the proposed footing-level, the lying material should be tested by one of the methods hereinafter describ

Testing with Steel Bars. A steel bar with a pointed end or a ste provided with a steel point is driven to the required depth by a maul o falling weight. While no samples can be obtained by this crude met may determine the ground-water level, and a little practice will enable distinguish sandy from clayey soils by the sound given out when the twisted. The difficulty of driving is a rough index of the degree of the pressibility of the soil. It should be remembered, however, that an material will afford considerable resistance to the bar and that a small b will stop it; so that not much reliance can be placed on a report that th DROVE HARD or that it REACHED ROCK.

* As codes change, quotations must be verified.

1. Loads in Tons per Square Foot on Foundation-Beds Allowed by Building Codes*

Character of foundation-bed	Richmond, Va.	Minneapolis, Minn.	Philadelphia, Pa.	Atlanta, Ga.	Portland, Ore.	Louisville, Ky.	[illegible]	[illegible]	Cincinnati, O.	St. Louis, Mo.	Cleveland, O.	San Francisco, Cal.
...vial soils					½						½	
...dry loam		3		2–3		2½	3					3
...clay	1	1		1			1	1				1
...ary clay								2				
...ately dry clay											2	
...d solid natural clay										3		
...y in thick beds, always dry									4			
...y in thick beds, moderately dry									2			
...y clay								3				
...dry clay	3	3		2–3		2½	3					3
...clay	4	4		†8 3–4		4	4	4				4
...y hard clay			3½		4						4	
...t wet clay and sand					1½							
...y sand mixed with clay								2				2
...ary clay and sand together ...layers; wet and springy	2	2		2			[illegible]					
...ately dry clay and sand					3							
...ified clay and stone									4			
...ksand					½						½	
...t wet sand											1	
...t sand								1				
...sand, firm and dry	3	3		2–3	4	2½	3					3
...dry fine sand									2			
...sand, compact and well cemented											4	
...y sand								3				
...compact sand									4			
...coarse sand								4				
...y firm coarse sand	4	4		3–4		4	4					4
...dry sand											2	
...l gravel	4			3–4		4	4					
...gravel		4		†8								
...ented gravel			6									
...d and loose gravel			3½									
...pact sand and gravel									5			
...pact sand and gravel, well cemented									8			
...coarse sand and gravel								6				
...el and coarse sand, well cemented					8						8	
...dpan							0–15					
...l shale, unexposed											8	10
...				†15	8						8	20

* Some values may change with the changes in building codes.
† In caissons.

Testing with Post-Hole Diggers. For shallow explorations in easi cavated material, the ordinary post-hole digger used for fence-posts, c longer and larger ones used for telegraph-poles, can be used to depths of 6 to 8 ft.

Testing with Augers. In clay or similar material a single or double carpenter's auger welded to a long rod, or the so-called POD-AUGER maj satisfactory samples. In gravel or loose and sandy material, the sides c hole fall in, clogging the operation and destroying the samples.

Testing by Dry-Pipe Borings. A POD-AUGER or the above-desc CARPENTER'S AUGER can be used inside a casing-pipe. The pipe shou driven so as to keep close to the bottom of the hole made by the auger. pipe prevents the material falling from the sides of the hole and the aug cavates and loosens the material ahead of the pipe and facilitates driving. above methods are not generally successful for deep holes or where g boulders or compact material interferes with driving the pipe.

Testing with Wash-Pipes. For test-borings over 10 ft in depth the m in most frequent use is the WASH-PIPE METHOD. In this method a wrough or steel pipe known as the CASING-PIPE or DRIVE-PIPE is driven into the ea much the same way as in the DRY-PIPE METHOD, but the driving of the p facilitated by the use of a JET OF WATER. The lower end of the casing-p provided with a hollow SHOE or reinforcement, slightly larger in outside dia than the casing. This serves to protect the pipe from injury in driving th gravel or hard-pan, and forms a hole slightly larger than the diameter c casing. The upper end of the drive-pipe is protected from injury by an an drive-head which has a threaded part fitting the thread on the casing-pip a central hole to admit the jet-pipe. The jet-pipe is small enough to p it to freely enter the casing-pipe. The lower end is contracted so as to pr a jet-action. The upper end is connected with a water-supply which mu under considerable pressure. The driving-mechanism consists of a cas weight with a central vertical hole large enough to admit the wash-pipe stationary verticals supporting a BLOCK-AND-FALL and an arrangement releases the weight when it has reached a predetermined height. With arrangement, water is continuously pumped through the jet-pipe, the leng which is regulated so that the jet-action loosens the material immediately l or AHEAD of the casing. Some of the jetting water returns to the surface side of the casing and thus lubricates the surface in contact with the ou material. Another part of the water returns to the surface in the an space between the wash-pipe and the casing, carrying with it particles o material loosened by the jet. As the jet loosens and washes away the ma immediately below the casing, the latter is driven deeper by repeated blo the ram, the driving and washing being carried on at the same time. operation is thus continuous until the top of the casing comes close to the face of the ground, when the hammer drive-head and hose-connection a moved to permit additional lengths of pipe to be added to the casing and w pipes, after which the hose-connection, drive-head and hammer are rep and the operation is resumed.

Borings can be made by this method to great depths in sand, clay or suitable material. Samples of the material encountered are obtained by s ment from the water returning between the jet-pipe and wash-pipe. T samples are not accurate samples as the water separates the materials. finer particles do not settle readily and the large and heavy particles may be brought up at all. It is evident that such samples do not give any i as to the solidity of the original deposits. If large gravel, hard-pan or bou

...ountered there will be great difficulty in forcing the casing past such ...tions. In such cases a DRILL-ROD is sometimes substituted for the JET ... obstruction broken up into small pieces or pushed to one side; but in ... case it is difficult to get any sample or real indication of the character ... obstruction. If solid rock or large boulders are encountered, no further ... can be made with the casing and no sample can be obtained by this ... Resort must then be had to one of the CORE-BORING METHODS described ..., to determine the character of the obstruction encountered.

...ing by Core-Borings. These borings can be made through rock or ... and accurate samples obtained. In all core-boring methods the hole ... by rotating a pipe-like tool which makes an annular cut in the rock ... a cylindrical core which is afterwards detached and brought to the ... by a gripping-tool called the CORE-LIFTER. The cutting is done in ... ways.

...mond Bits are annular rings fitted on the lower end of the hollow pipe ... the rotating drill-rod and furnished with a number of small diamonds ... so as to form cutting-edges, which, when rotated in contact with the ... gradually wear away the required annular space. The diamonds employed ... as BORT, BLACK DIAMONDS, or CARBONS, and their only resemblance ... stones used by jewelers is the necessary hardness. The carbons are ... secured in a soft metal bed, in sockets drilled in the bit, and they pro... ... the bit and also sufficiently inside and outside to insure the cutting ... large enough to provide clearance for the bit and the attached drill ... pipe.

...t-Drills. The same result is arrived at by the SHOT-DRILL METHOD, by ... particles of chilled cast iron called SHOT are used as the abrasive or cutting-... The shot is poured loose into the hole and forced against the rock by ... bit.

...ciency of Drill-Methods. Both of the drill-methods mentioned are ex... but as they are the only methods which will give an accurate sample ..., one or the other must be employed where the accurate determination ... is necessary. If the core corresponds to the known underlying rock-... and the rock is continuous for a length of from 8 to 20 ft, it is safe ... that solid rock has been reached. If, however, the core is of differ... ... from the known underlying formation, the probability is that a boulder ... encountered. If the core is not continuous it may indicate that ... seams in the rock or that there are detached rock-masses. The above-... methods are used after the overlying earth has been penetrated by ... the PIPE-SINKING METHODS previously described.

... Results of Pipe-Borings are frequently misleading and misinterpreted, ... care should be taken to compare the samples with samples obtained ... other borings where the exact character of the materials tested is known.

11. Loading-Tests

...eral Considerations. Loading-tests of the materials forming the foun-... ... are made to assist in determining its safe bearing capacity. It is not ... to what extent the supporting power of a given soil varies with the area ... to the unit load, and tests on small areas are not a safe guide for the ... on large areas. On account of the expense involved, tests on large ... rarely made, the usual test being on an area of about 1 sq ft. The ... be made on an undisturbed portion of the foundation-bed, leveled ... the test-load, and for a space around the area tested, so that the

adjoining material is not reinforced or SURCHARGED by a bank of unex
material. The load should be applied with the least possible jar or mo
of the surface in contact with the material of the foundation-bed.

Explanation of Methods. A convenient arrangement for this consists of a vertical timber or post carrying a platform to receive the te and having four horizontal guys at the top to keep the post in a vertical p The bottom of the post, forming the loading-area should be approximatel 12 in and its exact area should be known. The platform, sufficiently to support the load to be applied, should be concentric with the post close to the bottom of the post as practicable. The load may be pig iron, or sand in bags, or any other convenient material. The guys should be than four in number, should be attached to the top of the post and sho horizontally so as not to pull up or down on it. Levels should be re point on the post above or below the load, as may be most convenien load should be applied gradually and with the least possible jar, care bein also, to keep the loading uniform on opposite sides of the post, which sh always vertical. Levels should be taken at frequent intervals during the tion of the load. The level observed when the platform is first in positi be taken as zero and successive settlements referred to it. When the p unit load has been reached, no additional load should be added until no settlement is observed. After this, first 50 and later 100% overload added and the total and periodic settlements observed. If the settlemen a test-load of twice the proposed load is not excessive, the test is consider factory.

12. Topographical and Special Conditions

Excavations over Inclined Strata. In case the site of a proposed is on a slope, and especially if the slope is steep, there may be danger slip of the material forming the foundation-bed. (See, also, page 135. may occur if there is an inclined plane of separation between layers of th lying rock, or between the rock-surface and the material overlying t or if inclined strata or beds of clay occur below the foundation-bed. such locations are the more likely to occur if water is present, as th increases the weight of the soil and also reduces the COEFFICIENT OF F against sliding. Such conditions are frequently indicated by the appea springs or springy ground below the site. Where the base of the slope a stream or river there may be danger from the washing away of bank have been supporting the side slopes of the valley. In the case of deep with steep clay banks, or in any location where landslides have been k occur, great care should be taken to extend the footings to a bed that be affected by any landslide. It sometimes happens that there is a sl tinuous and general movement of the material forming the side slope of toward the center of the valley; but such conditions are rare, fortunatel general, no adequate protection is possible. In certain limestone fo there is danger from natural caves formed in the limestone by the a water.

Excavations Near Navigable Waters. When buildings are loca navigable waters, it not infrequently happens that dredging-operatio considerable distance induce a flow of fine sand or clay from strata un the adjoining banks. This has occurred where the existence of suc was not suspected. This danger is especially to be guarded against in localities adjoining waters which are, or may be, used as navigable st in locations near the water-front where it is likely that docks will be con

...age from Adjoining Excavations. Common and statute laws make ...l provision for the protection of property-owners against damage result... from the acts of others in making such excavations; but an owner has usu... control over such operations, whether on adjoining properties or streets, ... in general will prefer the assurance of safety to the possibility of damage ... building and the expense and uncertainty of a lawsuit. While it is not ... possible to guard fully against the effects of adjoining excavations, and ... the expense of so doing is not always justifiable, due consideration should ... to the matter. The following suggestions, therefore, may be of value.

...th of Adjoining Excavations. Footings adjacent to property-lines ...ated where there is a probability of future additions to a building, or ... of a building which adjoins property liable to become the site of building-...tions, should go down at least as deep as the maximum probable depth of ...jacent work. In estimating these probabilities, the character of the loca... should be taken into account. In medium-priced residential sections ... are rarely carried much deeper than 10 ft, a sufficient depth for a cellar ... height below grade. In high-priced residential sections it is not un... to have both a basement and a cellar, in which case a depth of cellar ... grade up to 20 ft may be expected. Cellars for residences are rarely ... below 10 ft, if in reaching that depth the excavation goes below the ...-level. In fact, a high water-level discourages deep excavation, not only ...count of the increased difficulty and expense of excavation but also on ... of the expense of waterproofing. In business sections, especially in ... of high ground-rents, there is an increasing tendency toward deep ..., especially in boiler-rooms, where clear heights of 20 ft and over are de... for modern water-tube boilers. The basements are frequently rentable ... figures for restaurants, vaults, stores, etc., so that in many instances the ... mechanical equipment of the building is housed below the basement in a ...ement and boiler-pit, the excavation for which extends down at least ... and in special cases 60 ft below the curb; and this notwithstanding the ... the water-level may be only from 10 to 20 ft below the curb.

...ers and Trenches as Affecting Foundations. In cities and towns ...ration should be given to the possibility of the construction of trenches ... streets. For the majority of localities it will be sufficient to consider ...bable depth of a sewer of the proper depth to serve the street. In other ... it will be necessary to consider the broader question as to the proba... of deeper excavations for trunk sewers, subways, etc. As such construc... are controlled by broad topographical considerations, no general rules can ... and the local city engineer should be consulted.

...dations Near Mines, Shafts, Wells, Etc. In mining-districts local ...ties should be consulted as to danger from the caving of OLD MINE-... No adequate provision can be made in the foundation against ...despread caving or subsidence as may result from mining-operations. ... cases, successive falls of rock-fragments from the roof may gradually ... voids left by the mining-operations, as the loosely piled fragments of the ... occupy more space as FILL than they did as part of the solid roof-mass. ...etimes happens that where the original working is deep, progressive ... of the roof fills all voids, and no surface-settlements result. In other ... overburden may settle as a solid mass, causing a settlement at the ... equal to the thickness of the old working. Precautionary measures may ... the filling in of the workings, a subject outside the limits of this chapter. ...se of an important building a local mining engineer should be consulted ...ssible, the location of the building changed to a safer site. MINING-

SHAFTS, DEEP WELLS, SHAFTS FOR TUNNELS, etc., may cause disturb the soil, but in such cases the settlement is generally concentrated aro shaft or well, and buildings at a reasonable distance are slightly affected.

Foundations Near Tunnels and Trenches for Railroads and Su In large cities the necessities of transportation are increasingly calling struction of underground RAILROADS, TUNNELS and SUBWAYS. Such const are generally planned to follow streets. Railroad tunnels for trunk li be expected to follow direct lines to centrally located stations or termina routes which avoid, as far as possible, difficulties of construction, conder of real estate and damage to high-priced properties. The depth of exc will generally be as shallow as practicable. Where the tunnel has to dip underneath some obstruction, the approach-grades will probably be at t imum or limiting grade of the particular section.

Relation of Subways to Foundations of the Most Important ings. In SUBWAY-CONSTRUCTION for rapid-transit passenger service, t can be operated on sharper curves and with steeper grades than would in the case of a trunk-line railroad. This permits the lines to follow the lines of the city streets. For traffic-considerations the locations general, follow the principal arteries of surface traffic, and stations will, eral, be located at intersections of important streets, where there is the congestion of population. As such conditions are caused by the exist trade-centers, and call for the construction of high buildings, it may re seen that the heaviest and most important buildings are most likely their foundations affected by the construction of a subway in their im vicinity. Where there is reason to apprehend the construction of su WAYS or TUNNELS, information should be sought as to the probable d the excavation, the depth at which water is encountered, the characte material, the probable width of the construction as affecting the use of s vaults, and the method to be employed in making excavations. Wh excavations for such tunnels and subways have been carried below th of the footings of adjoining buildings, as in Baltimore, Boston, Brooklyn, and New York City, buildings along the routes have been seriously a Such results have not been limited to any particular methods used in t struction of the tunnels, as even where the excavations were wholly, or in rock, serious damage has been done.

18. Loads Coming on the Footings

The Loads to be Considered in the design of the footings of a s are:

(1) **The Dead Loads,** or the loads due to the actual weight of the co structure, ready for occupancy.

(2) **The Live Loads,** or the loads due to the occupancy of the building to the weight of snow on the roof.

(3) **The Wind-Loads,** or the vertical components of stresses in the st produced by wind-pressure.

(1) **The Dead Load.** The dead load of any structure can be ac calculated. If the structure is properly designed the part of the de supported by each element of the foundation can be definitely stated. T dead load becomes effective as soon as the building is completed, and constant thereafter unless additions or alterations are made to or in th ture.

The Live Load. The live load of any structure is the sum of the roof- and floor-loads. In designing the roof and floors the calculations for are based on an assumed unit load which should be the maximum load, with the probable use of the structure, to which any portion of the floor may ever be subjected. The assumed live load is, therefore, probably greater than the average load for the entire area of a floor or the entire the roof. Moreover, as it is improbable that conditions of maximum will ever occur simultaneously on the roof and on all of the several it is probable that the maximum load on the footings will be less than the maximum loads on the roof and on the several floors.

Minimum Live Load for an unloaded building is zero.

Actual Live Load will vary from zero to a maximum, which maximum usually be less than the total assumed live load.

Ratio of the Probable Maximum Live Load to the Assumed Live Load in different buildings, so that no table or general rule can be given.

Probable Maximum Live Load. As it is important to know, approximately at least, the maximum live loads to which the footings will be subjected, this maximum may be only a fraction of the assumed live loads, the should make a careful study of the conditions of loading to which the will probably be subjected and estimate the probable maximum live for the entire building.

for Estimating Live Loads. (See, also, Chapter XXI, pages 718) In estimating the probable maximum live loads for different uses, the notes may be of value. In certain buildings the assumed unit load the roof and on parts of each floor may be reached at various times, is unlikely that the maximum loading of all parts of the building will at the same time. In buildings of many stories the probability of having loads on all of the floors at the same time decreases as the number increases.

Household and Office-Furniture weighs from 5 to 10 lb per sq ft of occupied. While safes, bookcases or filing-cases may produce local load- from 50 to 100 lb per sq ft, the average load on office-floors rarely reaches per sq ft.

Residences, Apartments and Parts of Hotels not used for public assemblies rarely loaded with more than 5 lb per sq ft of floor-area.

and Wholesale Stores require a large percentage of the floor-area for of salespeople and customers, and not over 50% of the floor-area is used storage of stock. In estimating the weight of miscellaneous stocks, an between the lightest and heaviest classes should be taken for the weight foot, and also, in figuring the total space occupied by stock, an average be taken between the maximum and minimum amount of stock carried. DRY-GOODS STORES the floor-load for the entire building may amount more than 25 lb per sq ft, but in WHOLESALE STORES, and especially in and hardware stores, the average load may greatly exceed this figure.

Workshops, Loft-Buildings and Buildings for Manufacturing, the actual will, of course, vary with the class of material handled and the weight machinery used, and no general estimate can be made. Where the character of the occupancy to be expected is known it is possible to make a close estimation of the weights of machinery, fixtures and average stock on each

Warehouses. In buildings used, in whole or in part, for STORAGE PURPOSES may be used for light, bulky materials which, when stowed so as to leave

gangways and working-spaces, will give a resultant load much belo
sumed load. On the other hand, the heaviest materials may be c
piled from floor to ceiling in defiance of building regulations, posted no
common sense. Raw materials or crated or baled materials can b
closer than miscellaneous articles, and are therefore liable to increase t

The Ratio of the Total Probable Maximum Live Load to the Total Live Load having been determined for the entire building, the probable n live load for any element of the footing may be readily obtained by mu the assumed or calculated live load for that element by this ratio.

(3) **The Wind-Load** is generally calculated on the assumption that may exert a uniform pressure, frequently taken at 30 lb per sq ft, on t external area of any side of the building. This assumption makes no d for the protecting influences of adjoining buildings. In a building of a is improbable that the maximum pressure will be reached over the entire area at the same instant of time, and consequently, if the assumed represents the maximum pressure, the average, at any time, will be l the calculated total.

General Effect of Wind-Pressure. The horizontal pressure of the wi to increase the load on footings on the leaward side of the building ar crease the load on footings on the windward side. In many buildings bracing, called wind-bracing, or other special construction, is used to the building from being deformed by the wind-pressure and to convert zontal stresses due to the wind-pressure into vertical components, acti defined lines of support, that is, into either uplifts or loads on certa piers or columns. Where the uplift on any element of the structure is the dead load on the same element, the uplift is ignored. Where the component increases the compression in any element it is called the WIND THAT ELEMENT of construction and on the corresponding footing. Th is generally based upon concentrating all of the wind-load on certain footings. If, on account of the general rigidity of the building, or on ac any other reason, the wind-stresses reach footings not designed to recei loads, the amounts figured on the external footings will be reduced cor ingly. It is probable that the maximum effect of the wind results from of impulses of short duration and that the effect of such pulsations partially overcome by the inertia and elasticity of the buildings; so resultant load reaching the footing may be only a part of the theoretical the instant during which the maximum pressure is exerted. (See, also, XXIX, Wind-Bracing of Tall Buildings.)

The Probable Maximum Wind-Load acting on the footing is, there than the theoretical load due to the maximum wind-pressure. If the wind-load represents approximately the maximum wind-pressure, as by a wind-gauge, it would appear safe to assume that only 50% of the wind-load would act to produce a settlement in the footings of a Some authors recommend that in proportioning footings all wind- ignored; but this, especially in the case of high and narrow buildings, festly improper. The minimum wind-load is negative, being actually from which the load may vary to the maximum, but the maximum reached only at rare intervals and will endure for a short period only.

The Combined Wind-Load and Live Load. It is improbable that t mum wind-load and the maximum live load will occur at the same ti consideration should be borne in mind when the estimate is being m the effective wind-load.

14. Assumed Loads Specified by Building Codes*

II. Requirements of Building Codes for Assumed Loads for Office-Buildings

City	Requirements
...a, Ga.	Live load, 75 lb per sq ft above 1st floor; 150 lb per sq ft on 1st floor Footings designed for dead load and 60% of live load and wind-load
...n, Mass.	Live load, 100 lb per sq ft. Wind-load, 30 lb per sq ft where erected in open spaces; in built-up districts, 25 lb at the 10th story, 2½ lb more for each succeeding upper story, up to a maximum of 35 lb to the 14th story and above
...lo, N. Y.	Live load, 70 lb per sq ft. Wind-load, 30 lb per sq ft. Foundations designed for the acting average loads in the completed and occupied building and not the theoretical or occasional loads
...apolis, Minn.	Live load, 75 lb per sq ft above the first floor; 100 lb for first floor. Wind-load, 30 lb per sq ft. Roof and top floor, full live load. For each succeeding lower floor, a reduction of 5% until 50% is reached, such reduction being used for the remaining floors
...mond, Va.*	Foundations designed for 60% of the live load
...ouis, Mo.*	Live load, 70 lb per sq ft; first floor, 150 lb. Loads carried by the soil, total dead load and 10 lb per sq ft of all the floor-area. Wind-load, 30 lb per sq ft
...Paul, Minn.	Live load, 60 lb per sq ft above the first floor. First floor, 125 lb. Wind-load, 30 lb. Roof and top floor, full load; for each lower floor, a reduction of 5% until 50% of the full live load is reached, when such reduced load shall be used for the remaining floors. Footings designed for dead load and live load
...innati, O.	Live load, 50 lb per sq ft above first floor; 100 lb for first floor. Live load reduced by 5% for each floor below the top until 20% is reached, when such reduced loads shall be used for remaining floors. Wind-load, 20 lb per sq ft above surrounding buildings
...go, Ill.	Live load, 50 lb per sq ft. 50% of the live load used for piles. Piers designed for 85% of live load on top floor and reduced by 5% for each lower floor until 50% is reached, when such reduced loads shall be used for the remaining floors. Wind-load 20 lb per sq ft
...York City*	Footings designed for 60% of the live load
...land, O.	Live load 60 lb per sq ft in offices proper. 100 lb per sq ft in halls, lobbies, etc. Footings for walls designed for 50% of live load. Free-standing columns designed for 80% of 100-lb load and 75% of 60-lb load. Wind-load 30 lb per sq ft; for free-standing structures in built-up districts 25 lb per sq ft at the 10th story and 2½ lb less for each lower story, and 2½ lb more for each higher story, until 35 lb is reached

...s are constantly changing. Richmond's new code gives floor-loads; St. Louis ...ed some values; New York City's new code gives floor-load values different ... of the former code.

Reduction in Assumed Loads. The building codes of various ci tain rules governing the assumptions to be made as to live loads and wi and these rules generally provide for some REDUCTION IN THE ASSUME Generally, it will be found possible to meet these requirements and at time arrange for the proper proportioning of the supporting areas. page 151, gives briefly the requirements of the building codes of several to assumed loads for office-buildings.

15. Proportioning the Supporting Areas for Equal Settlem

The Minimum Areas of Support. The actual dead loads and the live loads and wind-loads for each linear foot of wall and for each colu or other supporting element of the building down to the level of the having been calculated, a foundation-plan should be prepared giving th and center of action of all loads. For safety under the worst possible tion of loads, each footing should be ample to support the total of the de live loads and wind-loads coming on it. The MINIMUM AREAS OF SUP any footing are obtained by dividing the total of the dead loads, live l wind-loads by the safe supporting power of the foundation-bed. foundation-bed is rock, or can be considered as incompressible under load, the minimum areas so obtained may be used for the footings. pressible materials, or generally on all materials other than rock, th these minimum areas will not result in uniform settlements owing to that the actual live loads and wind-loads are not consistent with the live loads and wind-loads.

The Actual Loads on the Footings. In accordance with what previously said, let us assume that the dead load is constant, and th building under consideration the probable maximum live load is 50% assumed live load, that the probable maximum wind-load is 40% of the wind-load, and that on the completion of the building, for a short pe live loads and wind-loads reduce to zero. The ACTUAL LOADS ON THE would then be:

(1) Upon completion of the building, the dead load only;

(2) Under the maximum load due to occupancy and to snow on the dead load plus 50% of the assumed live load;

(3) When loaded as in (2) and subject, in addition, to the maximu able wind-action,

(*a*) The footings on the leaward side of the building will sustain dead load, plus 50% of the assumed live load, plus 40% o sumed wind-load;

(*b*) The footings on the windward side of a building will sustain dead load, plus 50% of the assumed live load, minus 40% o sumed uplift;

(*c*) Other footings will support the total dead load, plus 50% of sumed live load, plus zero wind-load;

(4) Intermediate conditions as to live loads and wind-loads will loadings intermediate between (1) and (3).

Variations in Unit Loads on Foundation-Beds. With such know tions it is, therefore, impossible to proportion the supporting areas so unit load on the foundation-bed shall be uniform at all times. If the ing areas are proportioned in the ratio of the dead load only, the buil completion, and before occupancy, will uniformly load the supportin and at that time all of the footings should show equal settlements; bu

[…], when the supporting areas have been subjected to the full effects of the […] and wind-loads, certain supporting areas, having a high percentage […] loads, or of live loads and wind-loads, will be subject to a higher unit […] and the corresponding footings will consequently settle more than other […] supporting a low percentage of live loads, or live loads and wind-[…].

[…]-Uniformity in Footing-Settlements. If, on the other hand, the sup[…] areas are proportioned on the basis of the dead loads, plus the maximum […], plus the maximum wind-loads, even if the MAXIMUM LOADS are the […] ACTUAL MAXIMUM LOADS, and not the FICTITIOUS ASSUMED LOADS, it is […] that upon the completion of the building and before occupancy, the […] areas having a lower percentage of live loads and wind-loads will […] higher unit load, and the corresponding footings will have settled more […] other footings supporting a high percentage of live loads and wind-loads. […] basis, the footings will not come to a uniform settlement until they […] subjected to the maximum live loads and wind-loads.

[…]itrary Rules for Proportioning Supporting Areas. Various ARBI[…] RULES have been recommended for the proportioning of the supporting […] to secure equal settlements. These rules generally provide for a reduc[…] the assumed live loads and wind-loads, but do not take into consideration […] that a large proportion of the total settlement of certain footings may […] place subsequently to the completion of the building and after other foot[…] have reached practically their full settlement.

[…]onal Rule for Proportioning Supporting Areas. The rule herein[…]commended provides not only for a reduction of the assumed loads on […] rational basis, but also for the proportioning of the footings for the mean […] instead of for the ultimate load, and it is believed that the resulting settle[…] will be as nearly uniform as possible. The rule is based on the propor[…] of the footings in accordance with the loads which will act on the […] at the time when all of the dead loads and one-half of the probable […] live loads and wind-loads exist. The reason for taking one-half of […] probable maximum wind-loads and live loads is that these loads vary from […] a maximum, the average being one-half of the maximum.

[…]vision for Variations in Loads. On the completion of the building […] before the live loads or wind-loads have gone on the footings, the settle[…] will not be uniform, because areas designed for a high percentage of live […] and wind-loads will have much less than their average load and will there[…] have settled less than footings having a low percentage of live loads and wind-[…]. When these same footings have been subjected to the maximum probable […] loads and wind-loads, the settlements will again be unequal, because the […] have been proportioned for only one-half of the probable maximum live […] and wind-loads; but the footings which originally were the highest will […] be the lowest. The inevitable movement due to the variation in the live […] and wind-loads will be equally divided, one-half of the settlement being […]ed to bring the footing to the level of a footing having the dead loads only, the other half of the settlement carrying it an equal distance below the same […]. In other words, the method provides for the least possible variation […] footings having different proportions of live loads and wind-loads.

[…] Mean Load. For lack of a better name, the loads taken for the pro[…]ing of the footings, consisting of the total dead loads, one-half of the […] maximum live loads and one-half of the probable wind-loads coming […] footing, will be called the MEAN LOAD.

The Mean Unit Load. The areas will be made such that the load foundation-bed due to the mean loads will be uniform, and this unifo which, in general, will be considerably less than the allowable unit load foundation-bed will be called the MEAN UNIT LOAD.

The Minimum Unit Load. The necessity for providing for th possible condition of loading is satisfied if the supporting area for all i is sufficiently large to support the total of the dead loads and the assun loads and wind-loads at the allowable unit pressure. The resulting a support are the MINIMUM AREAS, and any change in these areas neces make them proportionate to the mean loads must be effected by increasir areas rather than by diminishing any. Any mean unit load which wou when divided into the mean loads, areas, all of which would be larger t minimum areas, would serve as the mean unit load, but it is more eco to determine the LOWEST POSSIBLE MEAN UNIT LOAD which, when ap the mean loads, will give the least possible increase of the areas. This done by determining which one of the minimum areas carries the LEAS LOAD PER SQUARE FOOT. This area may be selected by calculating th load on each of the minimum areas, or more simply, by comparing th of assumed loads and a table giving the mean loads, and noting which has the LARGEST PERCENTAGE OF REDUCTION between the assumed load mean load. The resulting mean load on this footing will be the MINIMU LOAD which can be used as a MEAN UNIT LOAD.

The Method Reduced to Rule. The method can be reduced to follows:

(1) Prepare a table giving in vertical columns or table-divisions f footing, the dead loads, the assumed live loads, the assumed wind-loa the total of these three loads. This table is called the TABLE OF ASSUMED

(2) Prepare a similar table giving the dead loads, one-half of the ma probable live loads, one-half the maximum probable wind-loads and th of these three loads. This table will be called the TABLE OF MEAN LOAD

(3) By a comparison of the two tables, find the supporting area wh suffered the greatest percentage of reduction between the total assume and the total mean loads and find the unit load resulting from the mean l this area. This unit load will be called the MEAN UNIT LOAD.

(4) Divide the total mean load as given in the table of mean loads f footing by the mean unit load. The result will be the required AREA PORT.

Short Method for Determining the Mean Unit Load. From th going it follows that the MEAN UNIT LOAD can be obtained more directly by lowing rule. Find the supporting area which has suffered the largest per of reduction between the total assumed load and the total mean load and ply the allowable unit load on the foundation-bed by the ratio obtai dividing the total mean load by the total assumed load.

Illustrative Example. The following example is figured out more full is necessary in practice in order to fully explain the method and also to c the method with other methods frequently used and recommended. Ord the wind-loads on a building of the size and type assumed in the example w ignored, but they have been considered here to make the example compl

A factory-building (Fig. 2) is to have four floors above the basemen capable of supporting an assumed unit load of 200 lb per sq ft. The l the flat roof is assumed at 50 lb per sq ft. The horizontal wind-pressur sumed as a uniform pressure of 40 lb per sq ft, on the sides *AB* and *C*

tical component of the wind-pressure is to be taken care of by the foot-
f the side walls. There is also an interior self-supporting chimney and
ing shaft which is protected from the wind and which carries no floor-

undation-bed is a uniform, sandy material which is expected to compress
ly and at the rate of ½ in per ton of load per sq ft of supporting area.

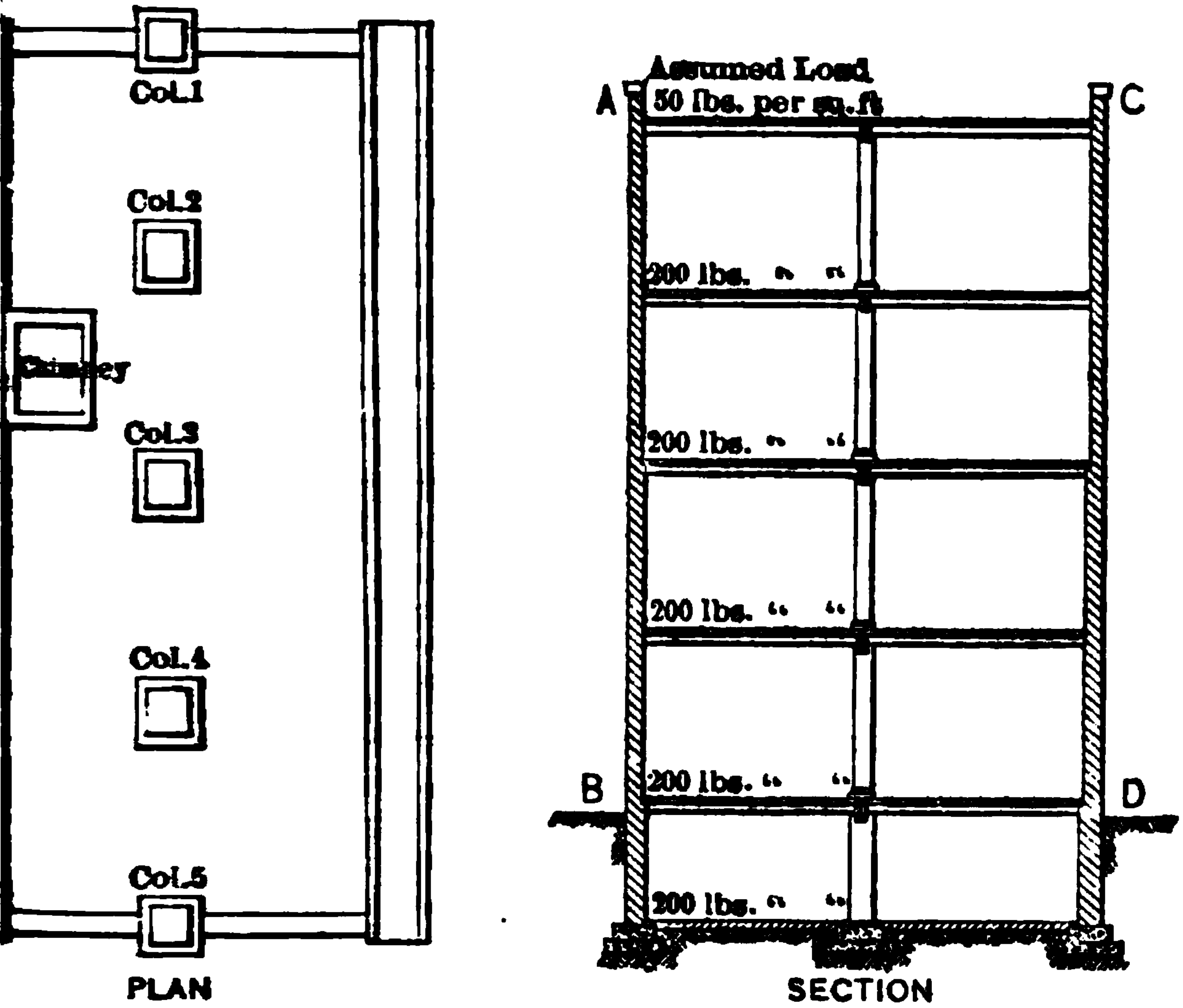


Fig. 2. Foundation-plan and Section of Factory-building

aximum unit load on the foundation-bed is taken at 4 tons, correspond-
a settlement of 2 in for the assumed load. The calculated dead loads of
uilding, including all construction down to the level of the footings, the
ation of the assumed live loads and the vertical components of the assumed
loads are given in Table III.

Table III. Dead Loads and Assumed Live and Wind-Loads

ement of footing	Division 1, dead loads only, lb	Division 2, assumed live loads, lb	Division 3, assumed wind-loads, lb	Division 4, total dead, live and wind-loads, lb
walls per lin ft....	14 000	8 400	2 000	24 400
mns 1 and 5......	137 500	160 000	....	297 500
mns 2, 3 and 4....	90 000	340 000	....	430 000
ney.............	320 000	....	....	320 000

columns are called divisions to avoid confusion with building-columns.

A careful study of the probable loading of the building shows that the ma
live loads at any one time will not exceed 60% of the total assumed liv
and that the maximum wind-loads will be less than 50% of the assume
loads, for the reason that the assumed wind-pressure is based upon the
recorded pressure on a limited area in an exposed situation, whereas t
posed building will be in a sheltered situation. Having, therefore, dete
the probable maximum live loads and wind-loads at 60% and 50% resp
of the assumed loads, the so-called mean loads, corresponding to loads h
between the minimum and maximum loads, will be one-half of the p
maximum loads, or 60% × ½ = 30% of the assumed live loads and 50
= 25% of the assumed wind-loads. Table IV gives the dead loads a
mean live loads and wind-loads separately, and the total of the dead loa
mean loads, which total is to be used in proportioning the areas for least
tion in settlement. This is known as the total mean load.

Table IV. Dead Loads, Mean Live and Wind-Loads and Total Dead Mean Loads

Element of footing	Division 5, dead loads, unchanged, lb	Division 6, one-half of 60% of assumed live loads, lb	Division 7, one-half of 50% of assumed wind-loads, lb	Divisi total loads
Side walls per lin ft....	14 000	2 520	500	17
Columns 1 and 5......	137 500	48 000	....	185
Columns 2, 3 and 4....	90 000	102 000	....	192
Chimney.............	320 000	....	....	320

Table-columns are called divisions to avoid confusion with building-columns.

Comparing the two tables it will be seen that the interior columns
building, columns 2, 3 and 4, had originally the largest percentage of liv
(no wind-loads), and have consequently suffered the greatest reduction
amount of total load. The minimum areas of support for columns 2, 3
and also for the other elements of the footings, are obtained by dividi
total assumed loads given in division 4, Table III, by 8 000, the all
unit load in pounds on the foundation-bed. The resulting areas are gi
division 9, Table V. No reduction can be made in these areas without e
ing the limitation that the most disadvantageous combinations of loading
ever improbable, shall not exceed the safe unit load. The adjustment
areas to the probable mean loading, as given in Table IV, the table of
loads, must be accomplished solely by increasing the sizes of certain footi

If we divide the total mean loads in division 8, Table IV, by the mir
areas given in division 9, Table V, we will get the mean load per squa
on the minimum areas for each element of the footing. The results gi
division 10, Table V, show that the mean load for columns 2, 3 and 4 i
3 568 lb per sq ft, while under the chimney the load is 8 000 lb per sq f
no reduction in area is permissible it is necessary to increase the footings
the chimney, side walls and columns 1 and 5 until the mean unit load
sponds to the mean unit load for columns 2, 3 and 5. This is done by di
the mean loads given in division 8, Table IV, by 3 568, the mean unit l

...ed for columns 2, 3 and 4. The resulting areas are given in division ... V, and are the areas which should be used.

...method of calculation can be shortened and reduced to a rule as follows. ... Table IV, the table of mean loads, with Table III, the table of assumed ... and find the element of support which has suffered the highest percentage ...tion between the total assumed load and the total mean load, and note ...sponding minimum area of support at the allowable unit load on the ...tion-bed. Divide the mean load for the same element of support by ...ber of square feet in the minimum area of support. The result will be ...it load for mean settlement. Then divide the mean loads for each ele... of support by the mean unit load. The results will be the required areas ... in Table V.

...le V. Mean Loads on Minimum Areas and Areas for Mean Loads

Element of footing	Division 9, minimum areas, sq ft	Division 10, mean loads on minimum areas, lb per sq ft	Division 11, areas for mean loads, sq ft
...walls per lin ft	3.05	5 580	4.7
...mns 1 and 5	37.2	4 986	51.9
...mns 2, 3 and 4	53.8	3 568	53.8
...ney	40.0	8 000	89.7

...e-columns are called divisions to avoid confusion with building-columns.

...he mean unit load may be determined by multiplying the allowable unit ... by the ratio obtained by dividing the mean load for the element of sup... having suffered the highest percentage of reduction by the assumed load ...e same element.

...ting Settlements. The following Tables VI, VII and VIII show the ...ative settlements which may be expected if the supporting areas are ...tioned in accordance with different assumptions as to load. In all the ... it is assumed that the foundation-bed will settle ½ in per ton of load ...uare foot, and that the total assumed load will never load the foundation-... excess of 4 tons per sq ft.

... Table VI the footings are proportioned in the ratio of the DEAD LOADS only. ... Table VII the footings are proportioned in the ratio of the TOTAL ASSUMED ...

... Table VIII the footings are proportioned in the ratio of the MEAN LOADS. ...ch table, division 1 gives the dead load coming on the footings on the ...tion of the building. Division 2 gives the load coming on the footings ... the building is subjected to the maximum probable live loads and wind-... Division 3 gives the supporting areas in accordance with the assumed ... Division 4 gives the settlements for the unloaded building. Divi... gives the settlement after the addition of the maximum probable live ... and wind-loads.

...ation of Table VI. The method of proportioning the areas in the ... dead loads only, as recommended by C. C. Schneider* may, in the form ... be stated as follows:

* ...rticle on the Structural Design of Buildings, Trans. Am. Soc. C. E., vol. 54, ...

Compare the table-division of dead loads, Table VI, with the div
assumed live loads, find the element of support which has the highest pe
of live loads to dead loads, and note the corresponding minimum area of
at the allowable unit load on the foundation-bed. Divide the dead
the same element of support by the number of square feet in this minim
of support, and the result will be the unit load due to the dead load only
divide the dead loads for all other elements of support by this unit lo
the results will be the areas required. Thus, in Table VI, it is seen by
to Table III that columns 2, 3 and 4 have the greatest percentage of
to dead load, and their minimum area of support, as in Table V, is 53
Then, 90 000 ÷ 53.8 = 1 675 lb, the unit load due to the dead load onl
area for columns 1 and 5 is 137 500 ÷ 1 675 = 82.1 sq ft. The process
lar for the other elements.

Table VI. Footings Proportioned in the Ratio of the Dead Loads (

Probable settlement where supporting areas are proportioned in the rati
dead loads only

Element of footing	Division 1 Dead loads only, lb	Division 2 Maximum probable loads, lb	Division 3 Areas, sq ft	Division 4 Settleme Empty, in	Di L
Side walls per lin ft.....	14 000	20 040	8.3	0.42	
Columns 1 and 5.......	137 500	233 500	82.1	0.42	
Columns 2, 3 and 4.....	90 000	294 000	53.8	0.42	
Chimney..............	320 000	320 000	191.0	0.42	
Maximum variation, empty....................			....	0.00	..
Maximum variation, loaded....................			....	....	

Table-columns are called divisions to avoid confusion with building-columns.

The calculations for settlements are readily made, when the amount
pressibility of the foundation-bed is known, by multiplying the unit load
foundation-bed of each element of support by the amount of compress
the foundation-bed per unit of load. Thus, in the above example the
of compressibility is given as ½ in per ton. In Table VI the unit load
dead loads for each element of support, are the same, or 1 675 lb = 0.
per sq ft, which, multiplied by ½ = 0.42 in. Similarly, the unit load
maximum probable loads for each element of support are determined,
loads, in tons, multiplied by one-half, give the settlements in inches as
division 5 of Table VI.

Explanation of Table VII. The areas given in Table VII are ob
dividing the total maximum dead loads, live loads and wind-loads (T
by the allowed unit, 8 000 lb per sq ft, and are the minimum areas given
V. The settlements for the loaded building are based on the maximum
loads as given in division 2 of Table VII.

Table VII. Footings Proportioned in the Ratio of the Total Assumed Loads

Probable settlement where supporting areas are proportioned in the ratio of total assumed loads

Element of footing	Division 1	Division 2	Division 3	Division 4	Division 5
	Dead loads only, lb	Maximum probable loads, lb	Areas, sq ft	Settlements	
				Empty, in	Loaded, in
Side walls per lin ft.....	14 000	20 040	3.05	1.15	1.64
Columns 1 and 5........	137 500	233 500	37.2	0.92	1.57
Columns 2, 3 and 4.....	90 000	294 000	53.8	0.42	1.36
Chimney..............	320 000	320 000	40.0	2.00	2.00
…imum variation, empty.....................			....	1.58	....
…imum variation, loaded....................			....	....	0.64

Table-columns are called divisions to avoid confusion with building-columns.

Table VIII. Footings Proportioned in the Ratio of the Mean Loads

Probable settlement where supporting areas are proportioned in the ratio of total mean loads

Element of footing	Division 1	Division 2	Division 3	Division 4	Division 5
	Dead loads only, lb	Maximum probable loads, lb	Areas, sq ft	Settlements	
				Empty, in	Loaded, in
Side walls per lin ft.....	14 000	20 040	4.7	0.74	1.06
Columns 1 and 5........	137 500	233 500	51.9	0.66	1.12
Columns 2, 3 and 4.....	90 000	294 000	53.8	0.42	1.36
Chimney..............	320 000	320 000	89.7	0.89	0.89
…imum variation, empty.....................			....	0.47	....
…imum variation, loaded....................			....	....	0.47

Table-columns are called divisions to avoid confusion with building-columns.

…planation of Table VIII. The areas in Table VIII are obtained as …ly explained and as given in division 11, Table V, and the methods used …termining the settlements are similar to those used for the preceding … In Table VIII it will be noted that columns 2, 3 and 4 have a settle… of 1.36 − 0.42 = 0.94 in, as a result of the addition of the live loads and …loads. Half of this settlement is required to bring these footings down to …level of the chimney-footing, and the other half of the settlement brings

them below the chimney-footing. There is no way to prevent this sett of 0.94 in, but its effect on the building is reduced to a minimum by hav settlement of the footings of columns 2, 3 and 4 start above the chimney- and finish below it. The chimney-footing does not change its elevatio the completion of the building, and compared with it, the variation in l the other footings is the minimum. In their mean position, half-way i movement, these other footings will be at the same level as the chimney-f

16. Determining the Supporting Areas

General Requirements. In laying out the AREAS OF SUPPORT for any ture it should be borne in mind, as previously explained, that (1) the t the dead loads, assumed live loads and assumed wind-loads should not lo foundation-bed in excess of the allowable load on it; (2) when the foun bed is compressible the areas of support should be calculated by the mea mean loads; and (3) the center of gravity of the supporting area should c with the center of action of the load to be supported. To these may be a further condition that (4) economy will be furthered by keeping the su ing areas simple in outline and by arranging each area as compactly as p around the center of the load to be supported.

(1) The first condition is necessary in order to provide that no possib dition of loading will exceed the allowable pressure on the foundation-be

(2) The second condition provides for making the settlements of di footings as nearly equal as possible.

(3) The third condition provides that the settlements of each footin be uniform, that is, that the footing shall not settle out of level.

(4) The fourth condition provides for economy in design in the footin and for economy in making the excavation for the footing, especially in t of deep excavations requiring sheathing for the protection of their sides.

In the case of a free-standing structure, the total load of which is not in of the supporting capacity of the entire area of the building at the sa load on the foundation-bed, it will generally be possible to arrange simp porting areas whose centers will correspond with the centers of the loads disposition of such areas is considered in succeeding paragraphs in the sions of CONCENTRIC LOADING. In buildings having restricted sites, walls or columns are placed close to adjoining property-lines, it will fre be impossible to arrange for simple concentric loadings and necessary offset footings, cantilevers or other devices to transfer the loads to sup areas located on the property. Such supporting areas are discussed in s ing paragraphs relating to ECCENTRIC FOOTINGS.

Footings with a Concentric Load. In order to have the load foundation-bed uniform under a footing it is necessary that the center c ity of the supporting area should coincide with the center of gravity of th otherwise the area is said to be ECCENTRICALLY LOADED and the resulti on the foundation-bed will not be uniform. Any variation in the loadi compressible foundation-bed under a footing will result in an unequal set of the footing and this in turn will result in unequal stresses in the wall, column supported by the area.

Wall-Footings with Concentric Load. In the case of a WALL, th should project an equal distance on each side so that the center of g the supporting area will coincide with the center of gravity of the wa the loads transmitted by the wall. The width of the supporting are vary with the load on the wall, irrespectively of any change in the thi the wall

ing for a Concentric Isolated Load. In the case of a SIMPLE CON-ED LOAD, as, for example, a load from a COLUMN or PIER, the footing be CIRCULAR, SQUARE, RECTANGULAR, or IRREGULAR in outline, but the of gravity of the area must coincide with the center of gravity of the load. mically the CIRCULAR SHAPE gives the most economical footing, as the ting areas extend radially the least possible distance from the center or of the load. Where deep excavation is necessary the circular form may itself to an economical method of excavation, as, for example, when cylin-l piers are sunk by the pneumatic method or by dredging. In general, ver, for ordinary footings the RECTANGULAR FORM is preferable, in that it itself to an economical arrangement of grillage-beams. The SQUARE is most economical rectangle as the sum of ending movements in the grillage and s is reduced to a minimum.

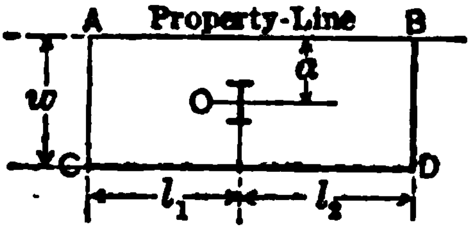


Fig. 3. Elongated Supporting Area. Concentric Load

ngated Supporting Areas. When the rting area for an isolated load cannot be a circle or a square, for example, when quare or the circle would overlap an ad- g property-line or interfere with an ad- g supporting area, the necessary area frequently be made RECTANGULAR in form, as *ABDC* (Fig. 3), having a h w, twice the distance a between the center of the load O and the ng line *AB*.

e required length l equals the required area divided by w and the area ld be centered on O, that is, l_1 must equal l_2.

mbinations of Simple Areas. Two Adjacent Isolated Areas. When ing supporting areas overlap or when, for other reasons, it is desirable to NE ADJACENT FOOTINGS, the best arrangement may be obtained as follows: ing the supporting area required for each of two adjacent concentrated and the distance between the centers of the loads, the sum of the two should be divided by twice the distance between the load-centers. The ent will be the width or the dimension of the required rectangle of sup- taken at right-angles to the line connecting the load-centers; and the dimension of the rectangle will be twice the distance between the load- s. The center of the area should be placed so as to coincide with the r of gravity of the two loads, when it will be found that each load will centric with its own area of support. Where a row of columns requires which nearly overlap, the COMBINATION OF THE AREAS frequently results in my in excavation and form-work.

porting Area for a Concentrated Load in the Line of a Wall. or more concentrated loads are carried in the line of a wall the ADDITIONAL TING AREAS required for such concentrated load may be provided in of two ways.

If the concentrated loads rest on the wall, as, for example, when the wall s the ends of girders and when the conditions are such that the con- ed loads are distributed along given lengths of it, then, all that is neces- s to increase the width of the footing for the given lengths sufficiently to for the total of the uniformly distributed and concentrated loads.

If a concentrated load is on the center line of the wall but cannot be dis- d by the wall, as when a considerable load is carried by a pier or column level of the footings, then one-half the additional area for the concentrated ould be placed on either side of the wall-footing, so that a line connecting ters of the two areas will pass through the center of the load. In general

it is desirable that the additional areas, together with the area for the wal between them, should APPROXIMATE A SQUARE. Knowing the width footing required to support the wall and the additional area required to s the concentrated load, the length of the side required square can be determined by the fol formula (Fig. 4):

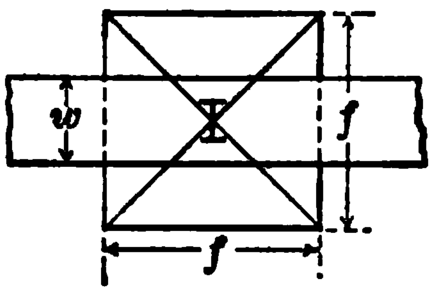


Fig. 4. Square Supporting Area. Wall and Concentric Isolated Load

Let w = the width of the footing;
A = the area required to support the c trated load;
f = the side of the square which will s a length of wall equal to f, an provide an additional area equa
Then

$$f = \tfrac{1}{2} w + \sqrt{A + \tfrac{1}{4} w^2}$$

Supporting Area for Concentrated Load not in the Center Lin Wall. The same additional supporting area is required for this as for centrated load on the center line of a wall, but the total area must be unequally between the two sides of the wall-footing, the larger portion being placed on the side of the eccentric load. The simplest way to determine the location of the supporting areas for this combination is to determine the size of the required square as if the concentrated load were concentric with the center line of the wall. The next step is to calculate the load due to the wall for the length of this square and determine the location of the center of gravity of the combined loads, that is, the center of gravity of this wall-load and the concentrated load. The center of the supporting area placed concentrically with the center of gravity of the combined load Fig. 5 let

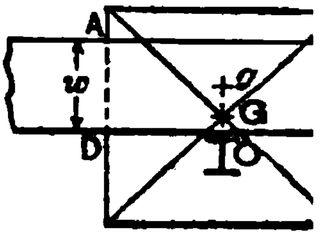


Fig. 5. Square Sup Area. Wall and tric Isolated Load

w = the required width of the wall-footing;
O = the concentrated load;
A = the area required for the support of the concentrated load.
as before, the length of the side of the required square will

$$f = AB = \tfrac{1}{2} w + \sqrt{A + \tfrac{1}{4} w^2}$$

The center of gravity of the wall-load contained between the lines A BC is at g, and the amount of the load is evidently the load per foot mu by the distance $AB = f$. Knowing the positi amount of the loads at O and g, the center of g of the combined loads is determined, say at G. fixes the center for the square.

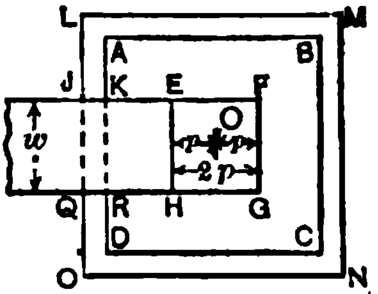


Fig. 6. Square Supporting Area. Isolated Load on End of Wall

Supporting Area for a Concentrated Lo the End of a Wall. A somewhat different tre is required for this, but the supporting area n best determined as follows (Fig. 6): Knowi width w of the footing required for the support wall, the additional area required for the conce load O and the distance p from the center of the c trated load from the end of the wall, proceed in this way. Determine the whose area corresponds to the sum of the areas required for the supp the concentrated load and for a length of wall equal to twice the projection wall beyond the center of the concentrated load. Plot this square $ABCD$

ation-plan and also the total area required for the support of the wall. square $ABCD$ includes an area sufficient for the support of the concentrated load for a section of the wall $EFGH$ corresponding to a length of wall equal to the projection p, multiplied by the width of the footing. It is evident the area $KEHR$ is loaded both by the wall and the concentrated load; in er words, that the square $ABCD$ is too small by the amount of the rectangle HR. The required square $LMNO$ will be approximately the square which contain the original area $ABCD$ plus the area $KEHR$, plus twice the area IQ. The length of the side $LM = MN$ will be approximately the length he original square plus one-half of the area $KEHR$ divided by the length he original square. The resulting square should be moved from the posi-shown on the drawing so that its center coincides with the center of ity of the combined concentrated load and the wall-load back as far as the re goes on the wall. A further approximation may be necessary where acy is required. The final result should be that the area of the square NO should be sufficient to support the concentrated load O and that portion he wall-load $JFGQ$ resting on the square, and that the center of gravity of square should coincide with the center of gravity of the combined loads.

17. Offset Footings

pporting Areas for Non-Concentric Loads. When walls, columns, or s are placed close to property-lines the required supporting areas cannot laced concentrically with the loads without lapping the property-lines. In such cases urse must be had to some method which will sfer the loads to supporting areas not con-ric with the loads. An attempt to accomplish result, the method known as OFFSETTING THE ING has been largely used, especially for side s adjoining property-lines. While theoreti-faulty, if not useless, it is indisputable that ET FOOTINGS have generally served the pur-for which they were designed. In the typical truction a cellar wall rests on a course of con-e or of flat stones forming a footing course iderably wider than the wall, the projection g entirely on one side of the wall. The load g on one side of the center of the footing s the supporting area unequally. The VARY-LOAD on the supporting area can be calculated llows: In Fig. 7 let

Fig. 7. Offset Footing. Varying Pressure on Foundation-bed

W = the total load per unit of length coming on the supporting area;
E = the eccentricity of load, that is, the distance between the center of the load and the center of the supporting area;
L = the width of the footing = the width of the supporting area = AB;
R_1 = the unit load, or pressure on the foundation-bed at A, the edge of the footing nearest the load;
R_2 = the unit load, or pressure on the foundation-bed at B, the edge of the footing farthest from the load;
y = any ordinate, from A to B.

hen the AVERAGE PRESSURE on the foundation-bed will evidently be W/L. essure at A, the edge nearest to the point of application of the load, will

be $K_1 = W/L\,(1 + 6\,U/L)$, or the MAXIMUM LOAD will equal the avera
plus six times the average load multiplied by the ratio of the ECCEN
divided by the width of the footing.

Similarly, the pressure at *B*, the edge farthest from the point of app
of the load, will be $K_2 = W/L\,(1 - 6\,U/L)$, or the MINIMUM LOAD equ
average load minus six times the average load multiplied by the ratio
eccentricity divided by the width of the footing.

When the ECCENTRICITY equals ⅙ the width, the pressure at *B* becom
If the eccentricity exceeds ⅙ the width there will be an uplift at *B*, or th
ing will have a tendency to overturn. This relation is generally expre
saying that to avoid an upward reaction the
line of the load must fall within the MIDDLE
of the base.

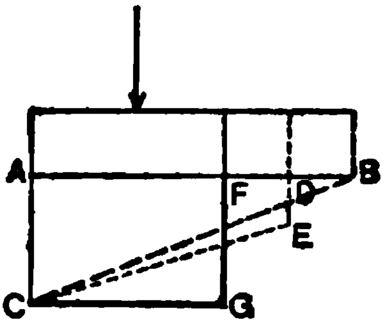


Fig. 8. Pressure-diagrams for Footings

Load-Diagrams for Offset Footings. In
diagram (Fig. 8) the figure *ADEC* represe
load-diagram on the foundation-bed for a w
footing *AD* and the load *AC* is the maximu
missible load, then the area *ADEC* represe
maximum support afforded by the footing *A*
the width is increased until the load falls on t
of the middle third or to the width *AB*, th
load at *B* is zero and the support is represented by the triangle *A*
area of which is less than the area *ADEC*. Moreover, if the w
the footing is reduced until its center is concentric with the load-cent
the load-diagram becomes *AFGC*, the area of which is greater than eith
or *ADEC*. From the foregoing it is evident that any advantage gai
offsetting the footing must be obtained at the cost of concentrating the
given to the wall away from the center line of the wall.

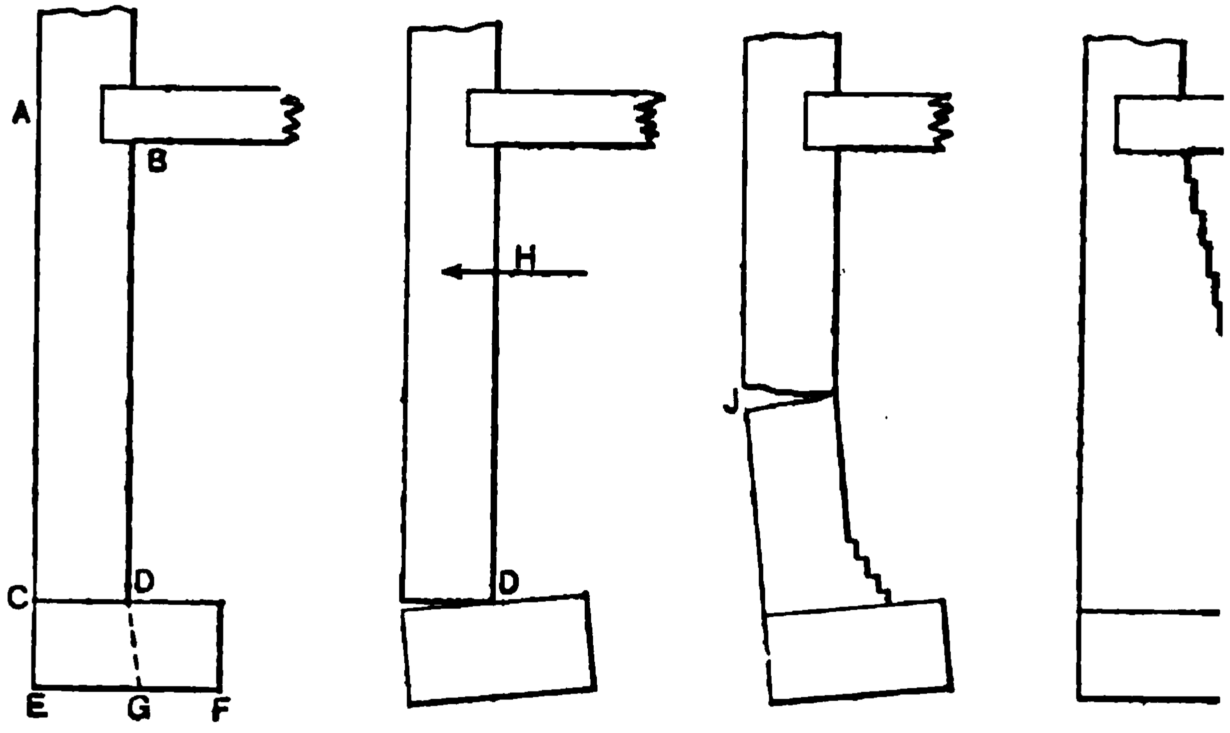


Fig. 9 Fig. 10 Fig. 11 Fig.

Figs. 9, 10 and 11. Eccentric Loading and Tendencies to Failure Due to Of
Footings; Fig. 12. Improved Type of Construction

Eccentric Loading Due to Offset Footings. In Fig. 9, represe
simple case of ECCENTRIC LOADING due to OFFSET FOOTINGS, the load
foundation-bed at *E* is perhaps twice the average load and at *F* abo
Under these conditions the projecting portion of the footing may shear,

along the line DG. If it does not shear and if there is any settlement to the load, the settlement will be unequal and the footing course will tend rotate into the position shown in Fig. 10. The entire load will then be transferred through the inner lower corner D of the cellar wall, rendering the wall stable and developing a tendency to move in the direction H.

The cellar wall may successfully resist this tendency by its own rigidity aided by the first-floor beams acting as ties or by the external resistance afforded by an abutting wall or bank of earth, or it may partially or completely fail developing a horizontal crack as indicated in Fig. 11 at J.

In this figure it will be noted that the base of the wall itself is offset. This is done to prevent the separate rotation of the footing course; but this construction does not diminish the TENDENCY TO ROTATION of the entire base of the wall and to the formation of a crack at J.

An improved type of construction is illustrated in Fig. 12, in which the floor-beams are anchored into the wall and the cellar wall has a continuous stepped offset from the level of the footing up to the level of the beams. The beams should evidently be arranged as tension-members, should run across the building and should be anchored in the opposite wall. While this method may have some effect it is of doubtful efficacy and should never be used for piers.

18. The Use of Cantilevers in Foundations*

Application of the Principle of the Lever. The use of the CANTILEVER, transferring a load to a supporting area not concentric with the load, is based on the PRINCIPLE OF THE LEVER and involves a girder or cantilever connecting two loads, and a supporting area or areas the CENTER OF ACTION of which is between the two loads. Part or all of the load on one side counterbalances the load on the other side of the center of the supporting area.

Illustrative Example. If an exterior column A (Fig. 13) carrying a load of 400 tons and requiring 100 sq ft of supporting area, at 4 tons per sq ft, the column-center being 18 in from a property-line PP which forms the limit of the building site, it is evidently impractical to employ a concentric footing 3 by 33⅓ ft for its support. If, however, a sufficient counterweight can be found in the shape of an adjacent interior column-load, as at B, the exterior load can be transferred by a girder or cantilever construction $CDEF$ to a supporting area MN lying between the two loads, and entirely within the limits of the property.

In Fig. 13 let PP represent the property-line, A the center of the load on column A, and B the center of the load on column B. Let the load on A be 400 tons, on B, 200 tons and the distance AB between centers, 20 ft. Assume that a rigid girder $CDEF$ supports and connects the two columns. If now a fulcrum or point of support G is provided for the girder at some point between A and B, the load on that point can be readily determined from the PRINCIPLE OF THE LEVER by multiplying the load on A, 400 tons, by the distance AB, 20 ft, and dividing the product by the distance BG, 19 ft; or, the load on G = $400 \times 20/19 = 421$ tons +. The area required for the support of this load, at 4 tons per sq ft, is $421/4 = 105\frac{1}{4}$ sq ft. The uplift at B, or the part of the load B required to counterbalance the overhanging load A is, from the principle of the lever, the product of the load A by the lever-arm AG divided by the lever-arm BG. The load on the footing for B is the difference between the original load and the uplift; but in view of the possibility of a reduction in the load A, which would decrease the uplift at B, it is well to provide for a possible increase in the load B.

* See, also, Chapter XIX, pages 678 to 680, for an example of a Continuous Girder in a Foundation.

Determination of the Area of Support. In determining the AR SUPPORT for A, having assumed one dimension of the supporting area twice the distance GP, or say 5 ft, the other dimension will be 105¼ sq ft/ ft ½ in. If the length 21 ft ½ in, as determined, is found to be excessiv

Fig. 13. Cantilever Foundation-construction

the point G must be moved to the left and the corresponding length of porting area must be determined as before. When the length of the sup area for the fulcrum of the cantilever is limited, so that the length pa the property-line is fixed, the width of the area can be determined experin or by the use of the formula

$$X = (L + u) - \sqrt{(L + u)^2 - 2\,WL/lS}$$

in which

L = the distance between centers of the two loads;
W = the load nearest to the property-line;
l = the length of the supporting area,
S = the unit load on the supporting area; and
u = the distance between the center of action of the load to be can and the edge of the supporting area nearest to the proper

If the position of the center of gravity of the load A combined w part of the load on B which is borne by the cantilever is determined, i found to coincide with the fulcrum or point of support G of the cantile demonstrating that the use of the cantilever provides a means of co two loads so that their center of gravity falls on the center of a suppor not concentric with either load.

The Grillage Fulcrum. Of course in practice the KNIFE-EDGE shown in the diagram is not used The bottom flange of the girder the cantilever rests on the DISTRIBUTING GRILLAGE directly, as is shown i which may be considered a typical arrangement.

The Girdering-Method for Two Equal Loads. When it is de support two or more adjacent concentrated loads on a single suppor

method called GIRDERING is employed. In the case of two concentrated ds, let A and B (Fig. 15) represent two columns. Let W_1 represent the load A and W_2 represent the load on B. Let D represent the distance between centers of the two loads. Let G represent the center of gravity of the comed loads. Let r represent the allowable unit load on the foundation-bed. e required area of support will be $(W_1 + W_2)/r$. This area may be of any

the distance D between the columns plus twice the extension e. Knowin length of the required area the width w is determined by simple division.

The Girdering-Method for Two Unequal Loads. In the case of col not equally loaded, the SUPPORTING AREA may be a TRAPEZOID, as in Fig the center of gravity of which must coincide with the center of gravity of loads. Knowing the sum and distance apart of the loads and the area for

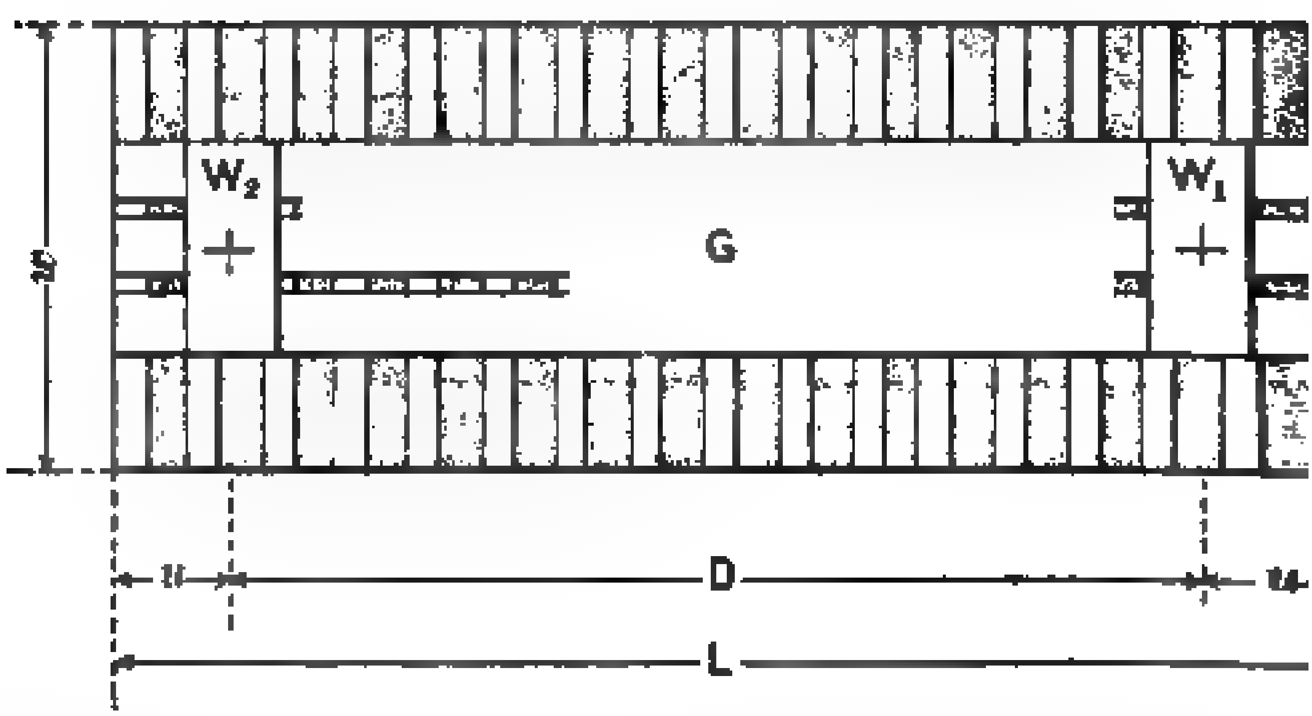


FIG. 15. Girdering-method of Foundations. Two Equal Loads

support, and fixing the total length L of the footing in accordance wi requirements that the footing shall not project beyond the line PP, the of the footing at the small and large end, a and b respectively, can be deter as follows: Let B represent the distance from the small end of the tra to the center of gravity of the two loads and let A represent the area trapezoid. Then

$$b = [2A/L] \times [(3B/L) - 1]$$
$$a = [2A/L] \times [2 - (3B/L)]$$
$$A = [(a + b)/2] \times L \quad \text{and} \quad a + b = 2A/L$$

For practical reasons the distance d should be made as small as possibl

Cantilevering an Exterior Wall. In the case of a wall the same principles apply, but the cantilevering effect must be distributed along the length of the wall. This can be accomplished by placing a girder under the wall, the girder in turn resting on the cantilever, or by using a number of cantilevers arranged

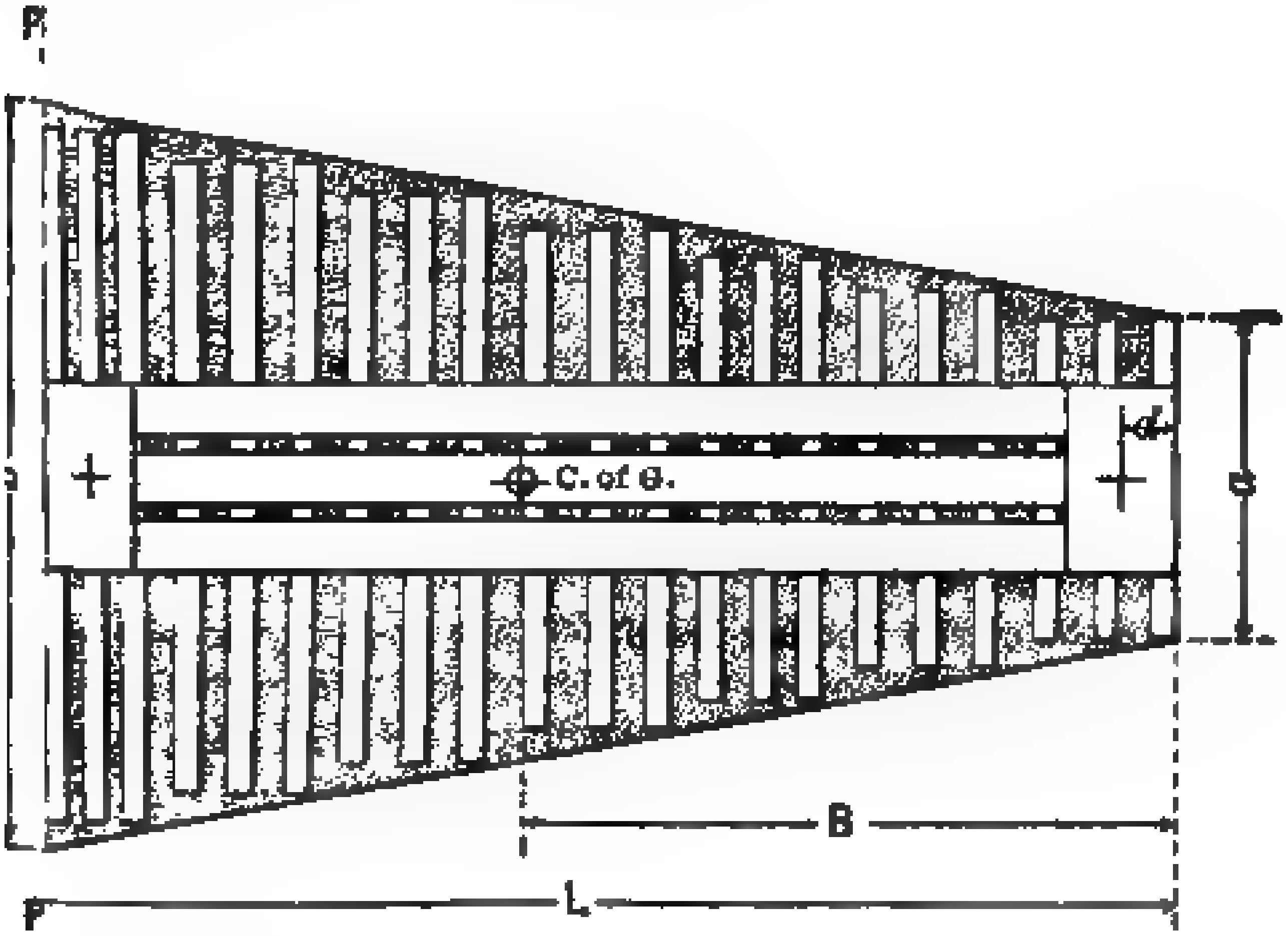


Fig. 16. Girdering-method of Foundations. Two Unequal Loads

in shape and radiating from the interior load-center. In narrow buildings cantilevers may run from wall to wall.

Double Cantilevering. The considerations controlling the design of the bearing areas required are the same as outlined in the preceding paragraphs.

10. Stresses in Footing Courses

Size and Form of Footing Courses. The footing courses of all walls and piers should be larger than the superimposed construction in order to secure STABILITY AGAINST OVERTURNING and to reduce the UNIT LOAD on the foundation-bed. When the change in size is accomplished abruptly as when a wall rests on a grillage or a slab of plain or reinforced concrete the footing is called a SPREAD FOOTING. When the base of the wall is thickened by means of offset

courses so that its bottom course is substantially as large as the footing the construction is known as a STEPPED FOOTING. It is evident that r and fast line can be drawn between the two classes. Whatever the form footing is it must be strong enough to distribute the more or less conce load coming on it, into a uniform pressure or load on the foundation-bed

The Unit Loads of Footing Courses. If the load on the upper of a footing course is uniformly distributed the intensity of the load, or i words the UNIT LOAD ON THE FOOTING, is obtained by dividing the total l the area of the base of the wall, pier, or other construction at that level load on the foundation-bed should be UNIFORMLY DISTRIBUTED and in the foundation-bed is compressible and the load concentric with the sup area, it may safely be assumed as uniform, since a compressible mater adjust itself until the loading at different points is substantially uniform unit load on the foundation-bed is evidently the total load divided by t porting area. If the area of the footing course varies between the top a tom of the footing the INTENSITY of the load will vary, and if uniformly distr the unit load at any level is obtained by dividing the total load by the the footing at that level.

The Weight of the Footing Itself. This is generally so small whe pared with the superimposed loads that it may be ignored without seriou

The Transmitting of Loads by Footings. If we neglect the we the footing we can consider the footing course as transmitting the impos to the foundation-bed or as being subject to two equal loads; one, the IMPOSED LOAD, more or less concentrated on the center line of the footi acting downward; the other, the REACTION due to the loading of the foun bed, uniformly distributed over the supporting area and acting upward. loads or forces being equal and opposite in direction, the stresses devel the footings are due to the differences in the distribution of these loa the footing courses simply act to convert concentrated into distributed l

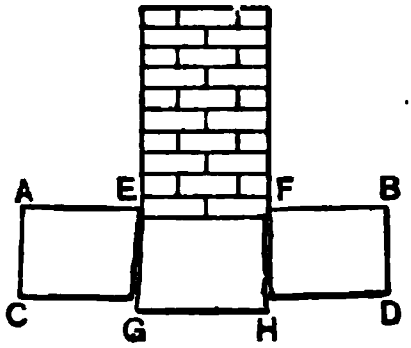


Fig. 17. Failure of Footing by Shearing

Manner of Failure of Footings. A footi fail in several ways: (1) by SHEARING; (2) by CRUSHING; (3) by SPREADING; and (4) by B OR RUPTURE.

(1) Failure of Footings by Shearing. ' illustrated in Fig. 17, showing a wall the we which has caused it to SHEAR along the lin and FH.

The force tending to cause SHEAR is the weig to the wall less the reaction of the foundation-b ing on the under side of the section $EFGH$. Si load is supposed to be uniformly distributed this is equivalent to the p of the area corresponding to the width CD minus the width GH times the of the wall considered, by the unit loading on the foundation-bed.

For a 1-ft length of wall the force causing shear, S, is

$$S = W\,(l - w)/l$$

in which W = the load due to wall per foot of length in pounds;
l = the width of footing;
w = the width of base of wall.

Or, since

$$W/l = U = \text{the unit load on the foundation-bed in pounds per squa}$$
$$S = U\,(l - w)$$

U is in terms of feet, l and w also must be in feet. The resistance to shear, under the conditions illustrated in Fig. 17, taken for a 1-ft length b of the is determined by the equation

$$R = 2 \times d \times b \times f$$

which f = the safe resistance of the material to shear, in pounds per square inch;
d = the depth of the footing in inches; and
b = the length of wall considered = 12 in.
ing $S = R$, we have

$$2\,dbf = U\,(l - w)$$

since $(l - w)/2$ = the projection of the footing

$$UP = 12\,df$$

depth of the footing, therefore, must not be less than

$$d = UP/12f$$

which P is in feet.

ar in Footings of Piers and Columns. FAILURE BY SHEAR is most likely occur in footings for piers and columns. The FORCE TENDING TO CAUSE is the total load on the column or pier less the reaction of the foundation-on the area immediately under the column-base. The resistance offered is ined by multiplying the perimeter of the column-base by the depth of the ing and by the allowable unit shear. When the area of the column-base all, the entire load may be taken as producing shear. When reinforced ete is used for the footing, there must be a sufficient number of stirrups to care of the shear. (See Chapters XXIV and XXV.) Where steel beams employed the cross-section of the beams must be sufficient to take care of shear, otherwise additional web-plates should be added, as is explained in ers XV and XX.

Failure of Footings by Direct Crushing. The failure of footings DIRECT CRUSHING of the materials composing the footings rarely, if ever, rs. Where, however, the concentrated load, due to a pier or column, is orted by beams or girders which have thin webs, the webs may fail by ling. Such beams or gird- should have their webs re- ced by vertical STIFFENERS y additional WEB-PLATES, and spaces between the beams ders should be filled with ete or grout. Where the transmitted by the column- exceeds the safe unit load the material of the footing area of the column-base may increased, or a block of granite be interposed between the ete or masonry footing and base of the columns. In case, however, such granite should be considered as a footing course and designed to resist bend- by formulas hereinafter given.

Fig. 18. Failure of Footing by Spreading

Failure of Footings by Spreading. Failure of the footings by SPREAD- may occur under walls or piers, as shown in Fig. 18, especially when the

foundation-bed is of clay or other yielding material, which has, under th of the footing, a tendency to FLOW along the lines indicated by arrows i figure. This tendency should be provided against by making the bottom continuous and adequate to resist the tension. Vertical joints, such a made in footings composed of masonry, are sources of weakness, and shou avoided. The TENDENCY TO SPREAD is greatest in footings having a spread is wide compared with the width of the superimposed wall or other constru The writer knows of at least one important footing which has failed in this the cracks in general following the joints of the masonry substantially as in Fig. 18.

(4) **Failure of Footings by Bending or Rupture.** A footing may f BENDING OR RUPTURE as a beam or girder. In the case of a wall, if the ing bends, as shown in Fig. 19, the concentration of the load on the lower

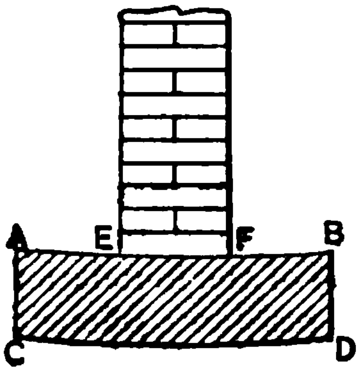


Fig. 19

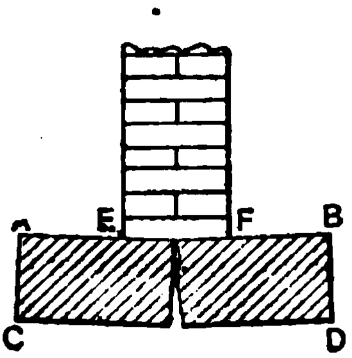


Fig. 20

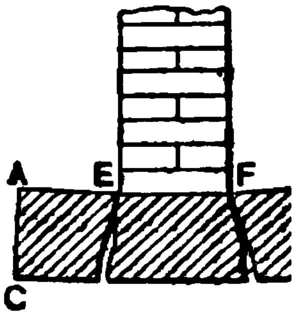


Fig. 21

Figs. 19, 20 and 21. Failures of Footings by Bending

of the wall, as at E and F may cause the base of the wall to fail. This poss should be borne in mind in designing footings where the load on the wa proaches the allowable unit load for the material composing it, and esp where the width of the footing is much greater than its own width.] footing fails by RUPTURE the rupture may occur either under the center l the wall, as in Fig. 20, or at points close to the outer edge of the wall Fig. 21. Fig. 20 illustrates the objection to using a footing course com of masonry or stones which do not extend the full width of the footing. joints in such construction prevent the footing course from acting in TE and the footing as a whole from acting as a BEAM.

20. Methods of Calculating Bending-Stresses in Wall-Footin

Assumptions Made in Determining Bending-Stresses in Foo

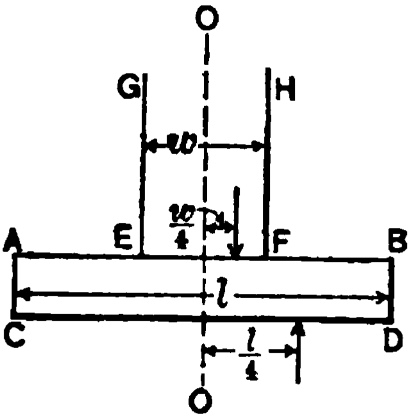


Fig. 22. Bending-stresses in Footings. First Method

Two methods for the calculation of the BEN STRESSES IN FOOTING COURSES are in genera Both are based upon the assumption tha REACTION of the foundation-bed is UNIFORM. the methods differ in the assumption made how the footing course and the base of the structure act. Neither assumption can be h be wholly correct.

The First Method of Determining Ben Stresses in Footings. This method is based the assumption that the pressure of the wall c footing is uniform over the area and remains so times.

If, in Fig. 22, $ABCD$ represents a footing course supporting a centrally l wall $EFGH$, and if

W = the load of the wall in pounds per linear foot;
w = the width of the wall in feet;
l = the width of the footing in feet;

$\frac{1}{2}(l-w)$ = the projection AE or FB,
$W/l = U$ = the unit load per square foot on the foundation-bed.

...idering the forces acting on the right of the center line of the wall for a ...ngth of wall, it is evident that the uplift on the half-footing OD will equal ...and that its CENTER OF ACTION will lie half-way between O and D, or at a ...ce $\frac{1}{4}l$ from the center line OO; and, similarly, that the load due to one-...the wall will be $\frac{1}{2}W$ and that its CENTER OF ACTION will be at a distance ...from the center line OO. The resulting moments will be

$$M_1 = \tfrac{1}{2}W \times \tfrac{1}{4}l = \tfrac{1}{8}Wl$$

$$M_2 = \tfrac{1}{2}W \times \tfrac{1}{4}w = \tfrac{1}{8}Ww$$

...s these two moments act in opposite directions, the resultant moment ...ing to produce bending in the footing will be the difference between the two, ...e bending moment at the center line OO is

$$M_0 = M_1 - M_2$$

$$M_0 = \tfrac{1}{8}W(l-w)$$

...ince

$$W/l = U \quad \text{and} \quad \tfrac{1}{2}(l-w) = P, \text{ the projection,}$$

...tion (1) may be written in either of the forms

$$\left.\begin{aligned} M_0 &= \tfrac{1}{8}U(l-w)l \\ M_0 &= \tfrac{1}{4}WP \end{aligned}\right\} \qquad (1)$$

...e **Error Involved** in this first method is due to the assumption that the ...re on the upper surface of the footing remains UNIFORMLY DISTRIBUTED, ...the base of the wall acted as a FLUID, in which case the distribution of the ...would remain constant and the formula would be correct. But the base ...e wall is not a FLUID, but a SOLID which will resist DEFORMATION. If, as in ...19, the footing course $ABCD$ deflects and the base of the wall is assumed ...t incompressible, the entire load of the wall will be communicated to the ...ng through the edges E and F. While such a concentration is, of course, ...ible (as the edges E and F will crush or com-... until a considerable area of the base of the ...is in contact with the footing) the result is that ...eight of the wall is concentrated near the outer ...s of its base. Equation (1) gives results which ...oo large; but as it errs on the side of safety, it ...mmended for general use.

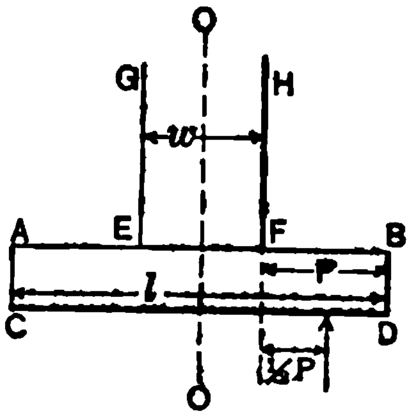


Fig. 23. Bending-stresses in Footings. Second Method

...e **Second Method of Determining Bending-...es in Footings,** also in common use, takes ...nsideration only the projecting portion of the ...g as follows:

...n Fig. 23 $ACBD$ represents a footing course ...ting a centrally located wall $EFGH$, and if ...e the notation of the preceding method, then, if we assume that the ...g acts as a FIXED BEAM and the projections AE and FB as CANTILEVERS ...supported by the wall, and denote the projection of the footing on either

side of the wall by P, the reaction of the foundation-bed on this pr
portion P, per unit length of wall, will be PU. The CENTER OF ACTION
force will be at a distance $\frac{1}{2} P$ from E or F and its moment at E or F

$$M = PU \times \tfrac{1}{2} P = \tfrac{1}{2} UP^2$$

or, since

$$P = \tfrac{1}{2} (l - w)$$

the value of M may be given in the form

$$M = \tfrac{1}{8} U (l - w)^2$$

The Error Involved in this second method is due to the assumption t
uplift on the projection P can be resisted by the extreme outer edge of t
of the wall. If the uplift on the projecting part is concentrated on th
then the edge must either compress or fail by crushing, which, in eith
would throw the center of support for the cantilever back from the edge
wall; and this is contrary to the assumption used in calculating the m
This method takes into consideration only the intensity of the reaction o
and the length of the projection, and is known as the PROJECTION-METHO

Comparison of Results. Comparing the results of the two methods,
be seen that the load cannot act at the two edges E and F as assumed in
tion (2), nor ordinarily can it be uniformly distributed as assumed in E
(1), but that the INTENSITY OF THE LOAD PER UNIT OF AREA will vary,
MINIMUM at the center and a MAXIMUM near the edges of the base of th
The exact positions of the CENTERS OF ACTION are affected by various co
ations which cannot be fully discussed in this chapter.

New Formula for Determining Bending Moments in Footings
writer has devised a formula which gives values for the bending mom
half-way between the values given by Equations (1) and (2), and which
corresponds to the assumption that, considering the forces on either side
center of the wall, the CENTER OF ACTION of the half-load of the wall is
center of the half-wall, when the projection equals zero, and, as the pro
increases, moves toward a position which is two-thirds of the distance fr
center of the wall to its edge. This formula may be expressed as follows:

$$M = \tfrac{1}{8} U (l - w) (l - \tfrac{1}{2} w)$$

Or, substituting the value of U in terms of W,

$$M = \tfrac{1}{8} W (l - w) (1 - w/2l)$$

Weight and Pressure-Units. In practice W, the weight due to the
generally given in pounds per linear foot of wall, and the allowable press
the foundation-bed, while frequently given in tons per square foot, sho
reduced to pounds per square foot.

The Required Width of the Footing in feet is obtained by dividing the
of the wall in pounds per linear foot of wall by the allowable unit load
foundation-bed expressed in pounds per square foot.

Moment-Units. The moment tending to produce rupture may be cal
in foot-pounds or inch-pounds. If in Equations (1), (2) and (3) the dim
l, w and P are in feet and U is in pounds per square foot, the resulting b
moment will be in foot-pounds per linear foot of wall. As the MOMENT OF R
ANCE is generally stated in inch-pounds it is more convenient to have th
IMUM BENDING MOMENT OR MOMENT OF RUPTURE * in inch-pounds. Th
Equation (1)

* In the flexure-formula the moment of resistance is made equal to the
moment at any cross-section of the footing, and the maximum bending mo
sometimes called the moment of rupture.

M (in inch-pounds per foot of wall) = 12 M in foot-pounds,

$$M \text{ (in inch-pounds)} = \tfrac{3}{2}\, U\,(l - w)\, l \qquad (1)'$$

...tion (2) in the same way becomes

$$M \text{ (in inch-pounds)} = \tfrac{3}{2}\, U\,(l - w)^2 \qquad (2)'$$

...ng the more convenient form,

$$M = \tfrac{1}{2}\, UP^2$$

...presses the projection P in inches, instead of in feet, we will have

$$M \text{ (in inch-pounds per foot of wall)} = \tfrac{1}{24}\, UP^2$$

...ly, Equation (3) becomes

$$M \text{ (in inch-pounds per foot of wall)} = \tfrac{3}{2}\, U\,(l - w)\,(l - \tfrac{1}{2} w). \qquad (3)'$$

...il Equations (3) or (3)′ are more generally accepted, an engineer or designer ...ound criticism and be perfectly safe in using Equation (1), and in the follow-...ges the writer will use Equations (1) or (1)′ unless the contrary is stated.

F

l in term of w

Fig. 24. Graphical Comparison of Bending Moments in Footings

...ple. The following is an example illustrating the application of the ...ing formulas:

...-in wall transmits to the footing 42 000 lb per linear foot of wall. The ...ble unit load on the foundation-bed is 3 600 lb per sq ft. What is the ...and required MOMENT OF RESISTANCE * of the footing?

$$42\,000 / 3\,600 = 11\tfrac{2}{3} \text{ ft}$$

* ...the flexure-formula the moment of resistance is made equal to the bending ...at any cross-section of the footing, and the maximum bending moment is ...so called the moment of rupture.

Then, by Equation (1), we have

$$M = \tfrac{1}{8} \times 3\,600\,(11\tfrac{2}{3} - 2)\,11\tfrac{2}{3} = 50\,750 \text{ ft-lb}$$

If Equation (2) is used, we have

$$M = \tfrac{1}{8} \times 3\,600\,(11\tfrac{2}{3} - 2)^2 = 42\,050 \text{ ft-lb}$$

and by Equation (3)

$$M = \tfrac{1}{8} \times 3\,600\,(11\tfrac{2}{3} - 2)\,(11\tfrac{2}{3} - 1) = 46\,400 \text{ ft-lb}$$

Comparing the results we see that the moment by Equation (3) is the
of the moments by Equations (1) and (2).

Graphical Comparison of Bending Moments in Footings. Fig
a graphical comparison of the moments for varying ratios of l to w cal
by Equations (1), (2) and (3) on the assumption that

w = the width of wall = 1 ft;
U = the unit load on the foundation-bed = 1 000 lb per sq ft; a
$r = l/w$.

The load on the wall, in pounds, for any value of l, is 1 000 l.

Comparing the curves of Equations (1) and (2) it will be seen that the
are widely apart, the percentage of variation being highest in the case
projections. When l is less than twice w, or in other words, when the pr
is less than one-half the width of the wall, Equation (2) gives moments le
half the moments given by Equation (1). Equation (2) may be used fo
projections. Equation (1) gives results which are too large, especiall
the projections are small. Equation (3), giving results half-way betwee
of Equations (1) and (2) and in accordance with a reasonable hypothesis
appear to be preferable, but is not in accordance with present practice.

21. Bending Moments in Footings of Columns and Piers

General Statement of the Problem. Fig. 25 represents in plan a
column resting on a footing which projects on four sides. The base
column or pier is represented by $ABCD$, a
footing and its area of support by $EFGH$.
part of the footing included in the areas MN
$QRST$ can be considered as acting in the sa
as projecting footings under a wall, but the u
the four corners $EQMA$, etc., on which no s
posed wall-load is imposed, also causes
moments.

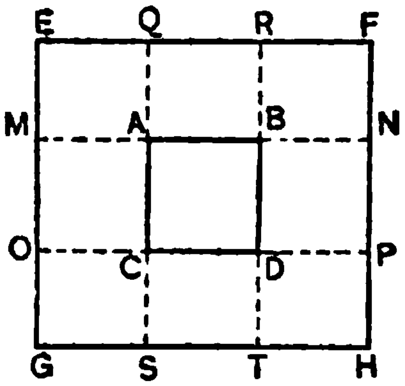


Fig. 25. Plan of Column-footing with Four Equal Projections

Different Theories. There are several t
more or less complicated and unsatisfactory
how the UPLIFT ON THE FOUR CORNER-AREAS
be determined. The discussion of these
would be out of place in this chapter. In a
footing, the projection is not over one-half the width of the superi
base, the four corner-areas will not aggregate over 25% of th
area of the footing, and it may then be assumed that the bending m
is the same as if the base of the column or pier extended like a wall
the entire footing, as is shown in Fig. 26. To insure these conditions, w
projection of the footing exceeds ½ w, and in all cases when the footing
homogeneous, as when a grillage of steel is used, the load of the colum
be distributed over the width of the footing by a GIRDER or BOLSTER o
extension of the column-base. In case the footing is in several layer

must extend the full width of the underlying layer. With such construc- it is evident that the bending moment will be the same as if the GIRDER or ... were a wall and Equation (1) will be applicable.

...ding Moments in Column-Footings. For column-footings Equation ... be used, taking the total load in place of the load per foot, and the result ...then be the total bending ...ent.

...mple. A column carrying ...s is to be supported on a ... concrete slab. The cast- ...column-base is 2 ft square. ...allowable pressure on the ...ation-bed is 6 tons per ... What is the MAXIMUM ...ING MOMENT in the slab? ...he area of support = 96/6 = ... ft = 4 by 4 ft. The pro-

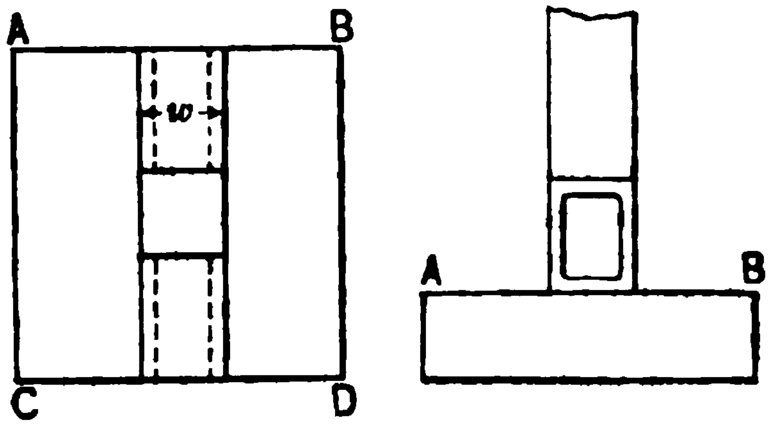


Fig. 26. Column-footing Treated Like Wall-footing

...n is ½ (4 − 2) = 1 ft, or one-half the width of the base, and by the fore- ... rule we can calculate the bending moment as if the base of the column ...ded in one direction across the footing. Applying a convenient form of ...tion (1)

$$M = \tfrac{1}{8} \times 192\,000 \text{ lb } (4 - 2) = 48\,000 \text{ ft-lb, or } 576\,000 \text{ in-lb}$$

...footing must therefore be of sufficient depth to resist this bending ...ent.

...n this example the allowable unit pressure on the foundation-bed is 2 tons ...ad of 6 tons per sq ft the supporting area and the area of the bottom con- ... footing course will be 96/2 = 48 sq ft. If the footing course can be a ...e its dimensions will be, with sufficient exactness, 7 by 7 ft. By the rule ..., since the projection exceeds one-half the width of the base, there should ... BOLSTER extending across the footing. The bolster will be, therefore, 7 ft ...and may properly be composed of two or more steel beams. The cast-iron ...may be dispensed with, in which case the base of the column will be pro- ...d with a steel base or with flange-angles. Let us assume that the column- ...is 1 ft 6 in square and the width of the bolster 2 ft.

...e bending moment in the bolster is determined, then, by Equation (1)· ... 1½ ft, the width of the column-base, for w, and 7 ft, the length of the ...r, for l.

$$M = \tfrac{1}{8} \times 192\,000 \,(7 - 1\tfrac{1}{2}) = 132\,000 \text{ ft-lb} = 1\,584\,000 \text{ in-lb}$$

...bending moment in the slab is determined in the same way by Equation (1), ... 2 ft, the width of the bolster, for w, and 7 ft, the length of the slab, for l.

$$M = \tfrac{1}{8} \times 192\,000 \,(7 - 2) = 120\,000 \text{ ft-lb} = 1\,440\,000 \text{ in-lb.}$$

...tings Other Than Square in Plan. In case it is necessary to use some ... shape than a square for the supporting area the resulting moments in ...slab and bolster will vary from those calculated above. If in the foregoing ...ple the supporting area, for any reason, is necessarily made 6 by 8 ft, giving ... ft as the required area, and if the bolster is parallel with the 6-ft side, the ...ent in the bolster will be

$$M = \tfrac{1}{8} \times 192\,000 \,(6 - 1\tfrac{1}{2}) = 108\,000 \text{ ft-lb} = 1\,296\,000 \text{ in-lb}$$

...he moment in the slab will be

$$M = \tfrac{1}{8} \times 192\,000 \,(8 - 2) = 144\,000 \text{ ft-lb} = 1\,728\,000 \text{ in-lb}$$

or, the moment in the bolster is less and the moment in the slab is grea
in the case of the 7 by 7-ft supporting area. If the bolster runs para
the long side, the moments will be, for the bolster,

$$M = \tfrac{1}{8} \times 192\,000\,(8 - 1\tfrac{1}{2}) = 156\,000 \text{ ft-lb}$$

and for the slab,

$$M = \tfrac{1}{8} \times 192\,000\,(6 - 2) = 96\,000 \text{ ft-lb}$$

In footings having more than two layers, each layer must be inv separately, using l for the length of the layer which is being determine for the width of the superimposed layer.

Compound Footings. In COMPOUND FOOTINGS where, for example and a column or two or more columns are supported by a single foo where loads are cantilevered, the loads will in general be distributed to t porting area by GIRDERS OR CANTILEVERS. The shears and bending m of such girders or cantilevers must be determined for each case by the m used in the calculations of beams and girders in Chapters XV and XX.

22. Design of the Footings

Materials used for Footings. To possess the required strength th MOMENT OF RESISTANCE of the footing must be at least equal to the m OF RUPTURE, calculated as explained in the preceding paragraphs. M whether of brickwork or stone, is not generally suitable for any but th est buildings, as its tensional strength is low. Concrete, plain or rein or grillages of steel embedded in concrete, are generally employed. (See ter III for footings for light buildings.)

Footings of Homogeneous Slabs. If the footing is composed of OF HOMOGENEOUS MATERIAL, as a block of granite or other reliable building or a single layer of concrete, the MOMENT OF RESISTANCE is, by the well- flexure-formula for rectangular cross-sections, $M_r = \tfrac{1}{6}\, bd^2 S$ (see Chapt XV and XVI) in which

d = the depth or thickness of the footing, in inches;
b = the breadth of the footing, in inches;
S = the allowable unit tensile stress of the material, in pounds per inch;
M_r = the moment of resistance.

Placing M, the moment of the forces tending to cause rupture, equal for a length of wall equal to 1 foot we have

$$b = 12 \text{ in}$$

and

$$d^2 = \tfrac{1}{2}\, M/S$$

Substituting in Equation (4) the value for M in inch-pounds as determi formulas (1), (2) and (3) and a value for S as given in the following para the required depth d can be determined.

Safe Tensional Strength for Materials in Footings. The value the ALLOWABLE UNIT TENSILE STRESS, for concrete or stone must include FACTOR OF SAFETY, as experiments show wide variations in the tensional s and in the MODULUS OF RUPTURE OR FLEXURAL STRENGTH of such ma The following values for S in pounds per square inch include a factor of of from 8 to 10 and should not be exceeded. (See, also, Table III, pa Chapter XVI.)

	S in lbs per sq in
...r brickwork or masonry in lime mortar.........	from 0 to 10
...r brickwork or masonry in cement mortar.......	from 10 to 40
...r concrete, 1 : 3 : 6..........................	from 15 to 25
...r concrete, 1 : 2½ : 5........................	from 20 to 40
...r concrete, 1 : 2 : 4..........................	from 30 to 50
...r sandstone or limestone in monolithic blocks....	from 75 to 150
...r granite in monolithic blocks.................	from 100 to 250

...ple of Concrete-Footing Design. Concrete Cast as a Unit. A concrete ...g course 4 ft wide supports a wall 2 ft thick. The load on the foundation ...28 000 lb per lin ft of wall, or 7 000 lb per sq ft. Assuming a value for *S* ...5 per sq in, what is the required depth for the concrete footing course? ...moment of rupture from one form of Equation (1)′ is

$$M = \tfrac{1}{3} W (l - w), \quad \text{or} \quad \tfrac{1}{3} \times 28\,000\,(4 - 2) = 84\,000 \text{ in-lb}$$

...tuting in Equation (4)

$$d^2 = \tfrac{1}{2} \times 84\,000/35 = 1\,200, \quad \text{or} \quad d = 35 \text{ in}$$

...uation (2)′ the moment of rupture is

$$M = \tfrac{1}{24}\, UP^2 = \tfrac{1}{24} \times 7\,000 \times 12 \times 12 = 42\,000 \text{ in-lb}$$

$$d^2 = \tfrac{1}{2} \times 42\,000/35 = 600, \quad \text{or} \quad d = 24 \text{ in} +$$

...pth determined by Equations (1) or (1)′, as previously noted, errs on ...e of safety. The result by Equations (2) or (2)′ conforms more nearly ...nal practice, and as the projection is small compared with the width of ...ll, it may be used, or an intermediate value, as determined by Equations (3)′. may be considered amply safe.

...ped Footings. If the concrete footing is cast in one uninterrupted ...n so as to act as a SINGLE GIRDER for its entire depth, a considerable ... of material may be effected by forming steps, ...wn in Fig. 27. If the steps are of equal ... the total projection should be equally divided ...n the steps. If the footing is cast in several ..., or if a granite slab is superimposed on a bed ...crete then each layer must be figured separately ...e width of the superimposed layer used in place ...he width of the wall.

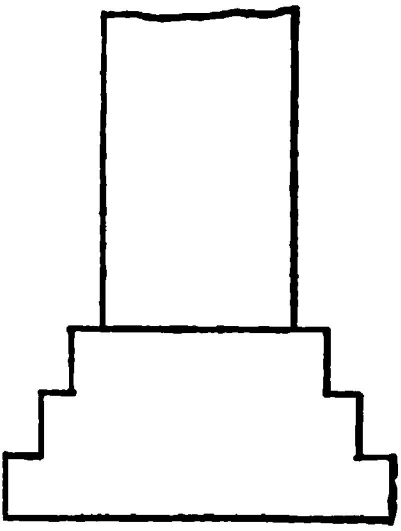


Fig. 27. Concrete Stepped Wall-footing

...ror in Design of Footings of Several ...s. Equation (2) should not be used where ...oting consists of several layers, as the error ... the erroneous assumption is cumulative and ... result in a serious concentration on the outer ...f the upper layers.

...ple of Footings of Several Layers. In the ...footings cast in separate layers the calculations ... be made as follows: Let l_1 = the length of the footing having a ...t, *M*. From Equation (1), reduced to inch-pounds,

$$l_1 = \frac{2\,M}{3\,W} + w$$

...g decided on the depth of each layer, say 15 in, and a value of *S*, say ...for concrete, then, from the flexure-formula, $M = M_r = \tfrac{1}{6} \times 12 \times 15^2$

$\times$ 35 = 15 750 in-lb, which, substituted in the above equation, will give th of l_1, or the length of the top course. Having determined l_1, the length second course, l_2, is found in the same way, using l_1 for w, and so on u required width of the footing is reached. The dimensions l and w ar taken in feet.

Comparison of Unit and Separate-Layer Footings. Footings i separate layers are very uneconomical in the amount of material require compared with those cast in one operation. If the footing in the p example is designed on the separate-layer basis and the courses assume 15 in thick, their lengths are as follows:

$$l_1 = \frac{2\,M}{3\,W} + w = [(2 \times 15\,750)/(3 \times 28\,000)] + 2 = 2.375 \text{ ft}$$

Also

$$l_2 = 2.75 \text{ ft}, \quad l_3 = 3.125 \text{ ft}, \quad l_4 = 3.50 \text{ ft} \quad \text{and} \quad l_5 = 3.875 \text{ ft}$$

As l_5 is nearly 4 ft, the required length, it may be made so by increas thickness of the bottom course to 16 in. The total thickness of the fo therefore (4 $\times$ 15 in) + 16 in = 76 in instead of 35 in, as previously dete by Equation (1) for the footing cast as a unit.

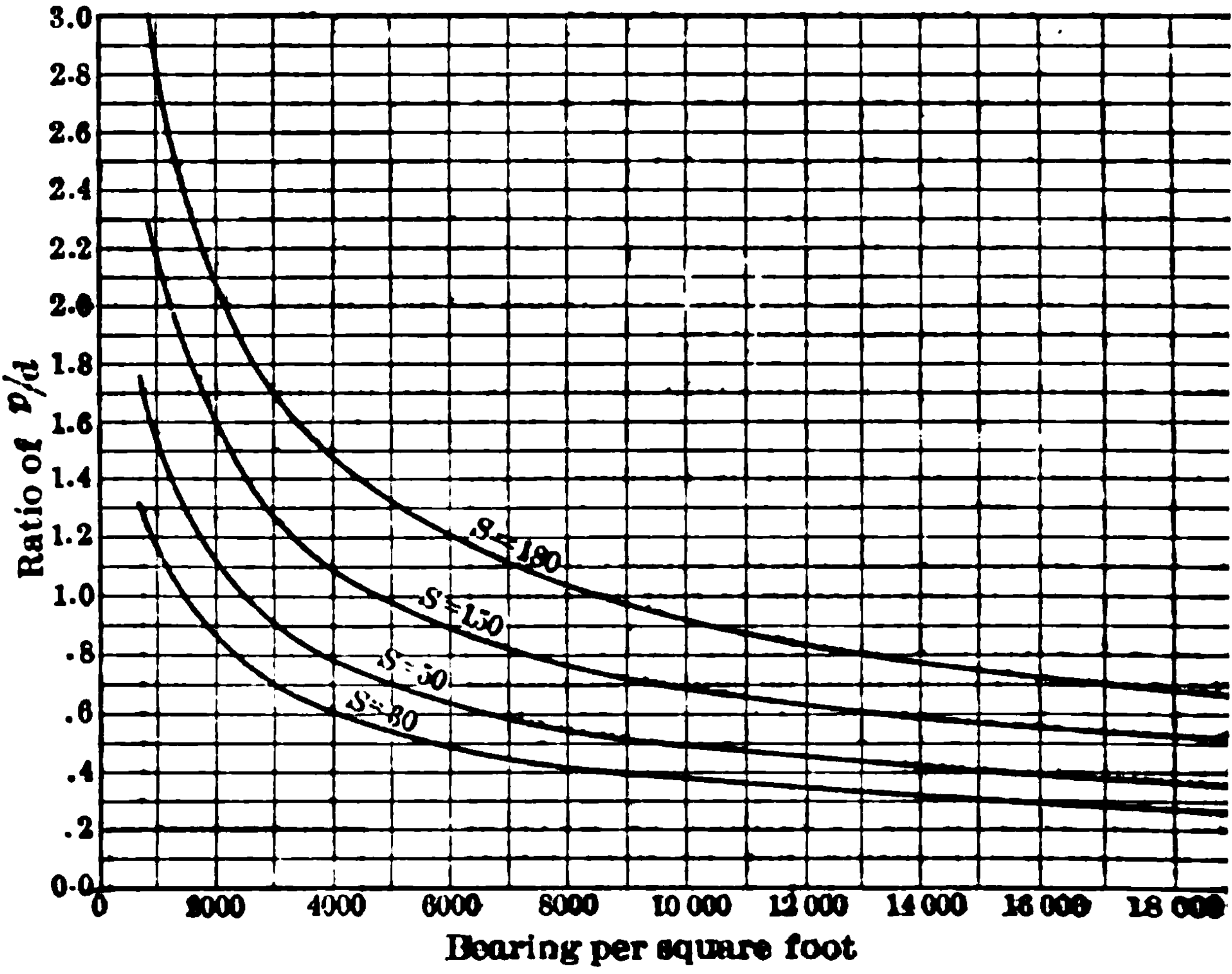


Fig. 28. Diagram Showing Ratio of Projection to Depth of Footings

Rule-of-Thumb Methods for Projections and Steps in Fo Various ARBITRARY RULES are in use which purport to give for different m of construction so-called SAFE PROJECTIONS for given depths of footin give the SAFE RATIO between the projection and the depth of a footing. rules ignore the fact that the uplift varies and they are entirely unreli though such RULES-OF-THUMB are often incorporated in the building c cities. (See Chapter III, page 224.)

Example. The safe projection for offsets in brickwork is frequentl in building codes and in text-books as 3 in for a double course of brick

of about 5 inches, the corresponding ratio being 0.6. If we assume the S for brickwork at 20 lb per sq in, this offset will be safe when the up-than 2 666 lb per sq ft, but not safe when the uplift is over 2 666 lb ft.

of **Projection to Depth of Footing.** For footings of homogeneous however, having a small projection and where Formula (2) can be used it is possible to calculate a so-called SAFE RATIO OF PROJECTION for a unit load. From Equation (2)′ and Equation (4), derived from the for the MOMENT OF RESISTANCE for beams of homogeneous material and cross-section, the following formula may be derived:

$$p/d = \sqrt{48S/U} \qquad (5)$$

all dimensions are in inches, S in pounds per square inch, and U in per square foot. The quantity p/d is the ratio of the projection to the of the beam or footing. For a given value of S the ratio will vary in- as the square root of U.

diagram (Fig. 28) shows curves for different values of S and U from which of projection to depth of footing may be taken. Thus, for a concrete for which the allowable unit stress, S, in tension is, say 30 lb per sq the load, U, on the foundation-bed is 3 000 lb per sq ft, the allowable pro- will be 0.69 times the depth of the footing course. If the concrete is thick, the allowable offset will be 8.3 in. Conversely, for a given offset, in, when the unit load is 3 000 lb and $S = 30$ lb as before, the required will be 1.45 times the offset.

23. Steel Grillages in Foundations*

ntages in the Use of Steel-Beam Grillages. When it is desirable id the deep excavation required for concrete or masonry footings, and the load of a wall has to be distributed over a wide area of support, STEEL or STEEL BEAMS are frequently advantageously used to give the required of resistance with a minimum of depth. Steel beams are generally and preferable to rails, although second-hand rails have frequently been as an expedient.

ring the Bed and Setting the Beams. The foundation-bed should covered with a layer of concrete not less than 6 in in thickness and so and compacted as to be as nearly impervious to moisture as possible. should be placed on this layer, the upper surfaces brought to a line lower flanges carefully grouted so as to secure an even bearing. Sub-ly, concrete should be placed between and around the beams so as to ntly protect them.

irements for Steel Grillages. In determining the number and size beams for any given footing the following points should be considered:

The beams must resist the MAXIMUM BENDING MOMENT, and this without DEFLECTION.

The beams must resist the SHEARING-STRESSES, the meeting of which ent ordinarily provides against CRUSHING.

The beams must not be spaced so far apart that there is danger of the filling between the beams failing to DISTRIBUTE THE LOAD.

The beams must not be spaced so near together as to prevent the placing ete between them. The clear space between the flanges of the top layer preferably be not less than 2 in and should be somewhat more for the yers.

* pages 678 to 680 for an example of a continuous girder in grillage foundation.

(5) Where the BENDING MOMENT is the governing feature, of two equal weight, the deeper beam should be used. Thus, if the required MODULUS is 147, a 20-in 81.4-lb beam might be used; but a 24-in 79-… is stiffer and stronger in bending.

(6) Where the SHEAR is the governing feature, of two beams of equ… the smaller beam is the stronger. Thus, the SHEARING VALUE of a 20… beam is greater than that of a 24-in 80-lb beam and is nearly equivale… of a 24-in 90-lb beam. However, on account of the greater STIFFNE… deeper beam it is sometimes advisable to use it even though the cost is…

Spacing of Beams in Grillage. Table IX gives the LIMITING SP… steel beams, based upon the safe capacity of the concrete filling a… beam, for loads of from 1 to 6 tons per sq ft. Since, however, in s… spans there is considerable ARCHING EFFECT, the concrete will safely… the load on larger spans than those given in the table, provided a… number of tie-rods of proper size are used to take up the THRUST of the…

Table IX. The Limiting Spacing for Steel Beams Used With Concre…

Depths of beams	Spacing of beams for the following pressures per square fo…									
	1 ton		2 tons		3 tons		4 tons		5 tons	
in	ft	in	ft	in	ft	in	ft	in	ft	in
6	1	3	0	11	0	10	0	9	0	8
7	1	6	1	1	0	11	0	10	0	9
8	1	8	1	3	1	1	0	11	0	10
9	1	11	1	5	1	2	1	0	0	11
10	2	1	1	6	1	4	1	2	1	1
12	2	5	1	10	1	6	1	4	1	3
15	3	0	2	3	1	10	1	8	1	6
18	3	8	2	8	2	3	1	11	1	9
20	4	0	2	11	2	5	2	2	1	11
24	4	9	3	6	2	11	2	7	2	4

The Design of a Wall-Footing of steel beams is illustrated by t… ing example: A 24-in wall carries 42 000 lb per lin ft. What should … and spacing of steel beams to distribute the load over the foundati… 3 600 lb per sq ft? The required width of the footing is 42 000/3 60… 8 in and the bending moment by Equation (3) is 556 800 in-lb pe… wall. The amount of shear, by the formula given on page 170, is $S = W$… or 34 800 lb. As the beams are in double shear the single shear per … of wall is 17 400 lb. The required section-modulus per linear foot of … tained by dividing the bending moment by the allowed fiber-stress i… or 556 800/16 000 (assumed fiber-stress) = 34.8. By referring to … page 355, giving the section-moduli of steel beams, we find that a 12… beam has a section-modulus of 36. To satisfy the condition of be… beams must not be spaced more than 36/34.8 = 1.03 ft, center to c… satisfy the condition of web-crippling due to direct compression, the … pressive stress must not exceed the value of S_b, Table II, page 5… for a 12-in 31.8-lb beam, is 13 060 lb per sq in. The area of the bea… compression is the length over which the load is distributed, times … thickness. Some authorities consider that the load is distributed ove…

the loaded portion of the beam plus one-half the depth of the beam, this and the following example the length of only the loaded portion is In this case the area is therefore 24 × 0.35 = 8.4 sq in. If the beams 1.03 ft on centers the unit direct compression is 42 000 × 1.03/8.4 = which is well within the allowed stress given by Table II, page 575. the condition of web-crippling due to shear, the shearing-stress must the value as derived from the formula for allowable shear. (See, XV, paragraphs and foot-notes relating to Buckling of Beam-Webs the illustrative Example 15 in that chapter.) The approximate, allowed, value may be obtained by dividing the value of S_b (Table II, 575) by the factor F, the values of which are given in Table IX A, For example, for a 12-in 31.8-lb beam this shearing value = 13 060/ 7915 lb per sq in. The shearing capacity of the beam is obtained by this unit stress by the depth of beam times the web-thickness, or 12 × 0.35 = 33 240 lb, or much more than required. Only one of the of web-crippling need be considered by applying the following rule: shear divided by the depth of the beam is greater than the total load by the product of the distance (over which the load is distributed) by F, investigate for shear; if otherwise, investigate for direct compres- This rule may also be expressed as follows: According as $(l - w)/l$ is greater than $2D/w'F$, investigate for shear or for compression. Here l = length w = loaded portion of beam, D = depth of beam, w' = length of beam which the load is assumed to be distributed (often taken = $w + \frac{1}{2} D$) and factor for the given beam obtained from Table IX A. All dimensions be taken in the same unit. If, instead of the 12-in beams, 15-in 42.9-lb having a section-modulus of 58.9 are used, the spacing will be 58.9/34.8 ft nearly, say 1 ft 8 in. By referring to Table IX, page 182, it is seen spacing of the beams is well within the safe limit of the concrete and no are theoretically necessary. It is preferable, however, to use at least of tie-rods.

Table IX A. Values of Factor F* for Shearing Values for Various Beams

Beams	For standard-weight beams	For heavy-weight beams
12-in beam	1.65	1.52
15-in beam	1.71	1.50
18-in beam	1.76	1.58
20-in beam	1.77	1.62
24-in beam	1.91	1.67

factors, F, which have been deduced to be used in connection with S_b, Table II, 574-5, to give the safe unit shearing value based on web-crippling, will help greatly computations of shears in case tables of safe shears are not obtainable. It is to be however, that the values derived from the use of F are approximate only, as this a little different for every beam; and to give its value for every beam would as much space as complete tables of safe shears. The values of F are not given sections of light beams as they are not usually good sections for grillages. be mentioned that the standard weight for each size of beam for which F is given the next weight higher than the minimum weight given in Table II, pages 574-5, the 20-in beams, for which the minimum weight, 65.4 lb, is also the standard. The rule given above for determining whether web-crippling based on shear or compression is the determining condition eliminates one of the calculations to in investigating grillages.

The Design of a Column-Footing of steel beams is illustrated following example: A column carries 576 tons. The allowable pressure foundation-bed is 3 tons per sq ft. What should be the arrangement, and size of the steel beams composing the grillage? The required area port = 576/3 = 192 sq ft. In order to make the problem as general as let it be supposed that practical considerations limit the width of the fo 12 ft. The dimensions of the concrete mat on which the lower layer of rests will be 12 by 16 ft. By referring to the diagram (Fig. 28) we find the mat is made 12 in thick an offset of 6 in is permissible. The dim of the lower layer of beams will therefore be 11 by 15 ft. A suitable gril the given conditions may be designed of two or of three layers. If tw are used the length of the top beams will be 11 ft. Assuming the colu = 30 in, the loaded portion = 2½ ft, and by Formula (1), the bending m = ¼ × 1 152 000 lb × (11 − 2½) × 12 × ½ = 14 688 000 in-lb, from wh required section-modulus (at 16 000-lb maximum fiber-stress) = 918. ferring to Table IV, Chapter X, five 24-in 90-lb beams have a section-m of 929 and consequently satisfy the condition of bending. By apply rule given in the preceding paragraph for the design of a wall-footing, web-crippling due to shear or to compression is to be investigated, $(l -$ 0.773 and $2\ D/w'F$ = 0.958, which, being greater than 0.773, shows that th should be investigated for web-crippling due to compression, by the explained in the previous example. It will be found that the five 24- beams also satisfy this condition and will therefore be used. Their width is about 7⅛ in, so they should be spaced about 9¼ in on centers ing the length of the column-base to be about 3 ft 9 in. The calcula the lower layer is similar, the length of the beams being 15 ft and the portion, 3 ft 9 in. It is rarely necessary to investigate the lower layer crippling, the condition of bending, except for the top layer, being usu governing feature. If, owing to conditions of bending, it is not practi make the beams of the top layer sufficiently long to extend across the width of the concrete mat, it is then necessary to make the grillage layers. The calculation for a three-layer grillage for the same problem preceding is as follows:

Calculation of the Top Layer. For web-crippling due to comp 1 152 000 lb = $S_b \times w' \times t \times n$, where S_b = the allowable unit stress, length of beam over which the load is assumed to be distributed, t = t thickness and n = the number of beams. Referring to Table II, Chap and assuming a 20-in 75-lb beam to be used, S_b = 13 660 lb per sq in 0.641 in. Taking w' = 30 in (the width of the column-base), 13 660 0.641 = 262 682 lb and the value for five beams is 1 313 410 lb, which than enough. But it is found that five 20-in 70-lb beams would not cient. It will be economical to make these beams of the greatest length f they will resist bending. The section-modulus of one beam is 126.3; total M_r = 5 × 126.3 × 16 000 (assumed fiber-stress). This may be dete also, by Formula (1) in which $M = \frac{1}{4}\,WP$. From these equations th tion P = 35¼ in, and the length of the beams is therefore (2 × 35¼) + width of the base) = 100½ in, or approximately 8 ft 4 in. By appl foregoing rule to see if web-crippling due to shear must be co (100 − 30)/100 = 0.7 which is less than 40/(30 × 1.62) = 0.82, and t need not be investigated.

* It is to be noted that the bending moment is the same as for a beam uniform with 576 tons on a span of 8½ ft, $(l - w)$, and that the number and size of the beams, as far as bending is concerned, may be taken from the tables giving the of beams. See Table IV, Chapter XV.

width of the flanges of these beams is nearly 6¼ in, so that they should be from 8½ to 9 in, thus making the required length of column-base 5 ft 6 in.

...ion of the Second Layer. Since the length of the top layer is limited ...4 in and the width of the lowest layer is 11 ft, it will be necessary to have ...mediate layer. This ...er the area given by ...gth of beams of the top ...and the width of the ...layer, or 8 ft 4 in by 11 ...he beams will of course ...right-angles to those of ...p layer, so their length ...11 ft, and they are to ...paced as not to exceed ...in. Since the width of ...course is 3½ ft, their ...is (11 ft − 3½ ft)/2 ...ft, the amount of single ...1 152 000 × 3.75/11 = ...lb and the bending ...is ½ × 1 152 000 × 45 ...960 000 in-lb. Using ...lb as the fiber-stress the ...section-modulus is ...by referring to Table ...apter X, for section- ...and determining the ...m shear as above ex- ..., we find that ten 15-in ...beams will have a total ...modulus of 812, and ...all also be ample for ... Furthermore, ten ...paced to cover a width ...in will give a spacing, ...to center of beams, of ...in, which is sufficient. ...ll be better, however, ...n 18-in 54.7-lb beams.

...tion of the Bottom ... Taking the effective ...f the middle layer as ...he projection of the ...(15 ft − 8 ft)/2 = 3½ ...n similarly to the ...he shear = 268 800 lb. ...2 096 000 in-lb, from ...the section-modulus = 756, and thirteen 15-in 42.9-lb beams, spaced 10½ ...ters, will be required, or two 15-in 60.8-lb beams and ten 15-in 42.9-lb ...ay be used, increasing the spacing between the beams. In this case ...y beams should be placed nearest to the center of the footing. This ...illustrated in Fig. 29.

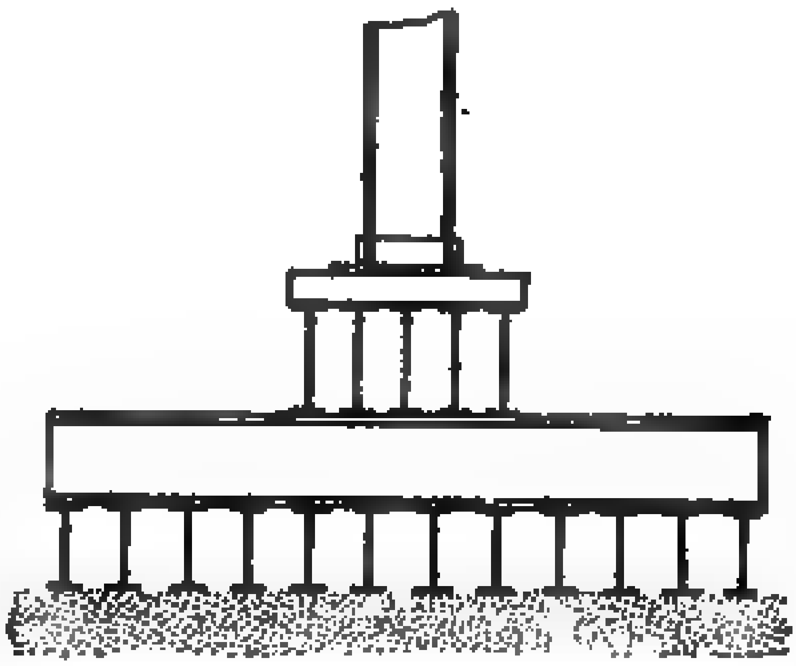


Fig. 29. Steel-beam Grillage Column-footing

24. Reinforced-Concrete Footings

Advantages and Disadvantages. Reinforced concrete has in recen been largely used for footings. The arguments in favor of its use are:

(1) Low cost of the footing-construction;

(2) Reduction in the amount of excavation required,

(3) Convenience, as compared with the use of steel-beam grillages, the reinforcing-steel is readily obtainable, can be cut to length on the w handled without derricks.

The objections urged are:

(1) Danger of defective workmanship, as the strength of the footing upon the proper mixing and placing of the concrete, the proper placin reinforcement and the complete union of the concrete with the reinfor The danger of defective workmanship is increased by reason of the usual ties of foundation-work, in that water and mud are generally present difficulty of careful work and inspection is greater.

(2) Danger of the deterioration of the steel reinforcement either by or by electrolysis. This danger is increased by the presence of moisture the relatively small cross-section of the reinforcing-bars. In this con it is well to remember that in reinforced-concrete girders as usually the concrete on the tension side is stressed beyond its elastic limit, as a which, numerous fine cracks are developed under the figured load.

Use of Reinforced Concrete for Foundations. From the foregoi apparent that great care should be used in connection with reinforced in foundations, especially as any defect is difficult to detect or repair. forced concrete is used not only for so-called MATS or SLABS but is fre used for DISTRIBUTING-GIRDERS, BOLSTERS and even for CANTILEVER author's preference is against reinforced concrete for foundations for im structures.

The Methods Used in Calculating the Strength of Reinforce crete Slabs, Girders, etc., are explained in Chapters XXIV and XX stresses coming on the reinforced-concrete construction are to be det in the same way as explained for footings of other materials.

25. Timber Footings for Temporary Buildings

Timber Footings. For buildings of moderate height timber may to give the necessary spread to the footings, provided water is always The footings should be built by covering the bottom of the trenche should be perfectly level, with 2-in planks laid close together and longit with the wall. Across these planks heavy timbers should be laid, spac 12 in on centers, the size of the timbers being proportioned to the tr stress. On top of these timbers again should be spiked a floor of 3-i of the same width as the masonry footings which are laid upon it. A of such a footing is shown in Fig. 30. All of the timber-work must below low-water mark, and the space between the transverse timbers s filled with sand, broken stone, or concrete. The best woods for such tions are oak, long-leaf yellow pine and Norway pine. Many of the o ings in Chicago rest on timber footings.

Calculations for the Sizes of the Cross-Timbers. The sizes of t verse timbers should be computed by the following formula:

$$\text{Breadth in inches} = \frac{2 \times w \times p^2 \times s}{d^2 \times A},$$

ing the bearing resistance of the foundation-bed in pounds per square the projection of the transverse timbers beyond the 3-in planks, in feet, nce on centers of the timbers in feet, and d the assumed depth of the n inches. A is the constant for strength.* The values recommended for for long-leaf yellow pine and white oak, 44 for Norway pine, and 39 for white pine or spruce, all increased from 30 to 40% for temporary build- (See Table II, page 628.)

Fig. 30. Spread Footing of Timber

ple. The side walls of a given building impose on the foundation a of 10 000 lb per lin ft; the soil will only support, without excessive nt, 2 000 lb per sq ft. It is decided for economy to build the footings as in Fig. 30, using long-leaf yellow-pine timber. What should be the size nsverse timbers?

n. Dividing the total pressure per linear foot by 2 000 lb, we have the width of the footings. The masonry footing we will make of granite hard stone, 4 ft wide, and solidly bedded on the planks in Portland, mortar. The projection p of the transverse beams will then be 3 ft. pace the beams 12 in on centers, so that $s = 1$, and will assume 10 in depth of the beams. Then, by the formula,

$$\text{the breadth in inches} = \frac{2 \times 2000 \times 9 \times 1}{100 \times 90} = 4 \text{ in}$$

hould use 4- by 10-in timbers, spaced 12 in on centers. If spruce timber d we should substitute 55 for 90, and the result would be 6½ in. (See p 628, for A increased from 30 to 40%.)

ations for Temporary Buildings. When temporary buildings are lt on a compressible soil, the foundations may, in some parts of the be constructed more cheaply of timber than of any other material, and ases the durability of the timber need not be considered, as when it is will last two or three years in almost any place, if thorough ventilation ed. The World's Fair buildings at Chicago (1893) were, as a rule, d on timber platforms, proportioned so that the maximum load on the d not exceed 1¼ tons per sq ft. Only in a few places over MUD-HOLES foundations used.

lues given to the term A of the formula vary in different building codes.

26. General Conditions Affecting Foundations and Footin

General Considerations. Where the footings of a building rest on w or on clay, it is important that any movement of the material forn foundation-bed be prevented if possible. In many cases it is advisable nect all footings with a concrete floor to prevent any UPLIFT of the fou bed between the footings. Where unequal settlement is apprehend inadvisable to have long columns firmly attached to the footings, as any settlement of the footings develops a BENDING-STRESS in the colum bending-stresses, in the case of long columns, may become extremely resulting possibly in the rupture or distortion of the columns. In such has even been proposed to design the bases of the columns with BALL-ANI joints which would allow unequal settlement of the footings without d or bending of the columns. Such connections, however, could not be g used because of the necessity of bracing the structure against the h pressure of the wind, but they would be entirely practicable in the cas interior columns.

The Minimum Depth of Footings is limited by the depth of th by the requirements of the cellar as to whether part of the footings car above the cellar-floor level, and by the depth of the footing itself. The n depth will be advantageously exceeded if, by a slight increase in depth, a capable of sustaining a higher unit load is found on which to rest the or if, as explained in previous articles of this chapter, greater security is by locating the footing at a greater depth. These considerations will the design of a footing and in all cases should be taken into considerat some cases it may be cheaper to abandon the use of a SPREAD FOOTING of and resort to PILES or MASONRY CONSTRUCTION going to ROCK or to so solid substratum. Where there is any question on this point, careful co should be made of the advantages and costs of the two methods. In however, it will be cheaper to spread footings immediately below th excavation level than to employ any of the various deep-foundation me

Deep Foundations are necessary when the material at the level SPREAD FOOTINGS would ordinarily be constructed is not suitable, or i is desirable for any reason to carry the foundations of the building do underlying stratum of greater supporting power. Recourse must the to one or more of the following expedients:

(1) Wooden piles;
(2) Concrete piles;
(3) Piers or walls constructed in pits or trenches, or by other meth going down to the required depth to reach a solid stratum.

27. Wooden-Pile Foundations

The Use of Wooden Piles. When it is required to build upon a sible soil that is constantly saturated with water and of considerable d most practicable method of obtaining a solid and enduring found buildings of moderate height is by driving wooden piles. Many bu the city of Boston, Mass., and several tall office-buildings of New Y and Chicago, rest on wooden piles, and they are extensively used for s buildings, grain-elevators, etc., erected along the water-front of coast cities. The durability of wooden piles in ground constantly satur water is beyond question, as they have been found in a perfectly sou tion after the lapse of from six to seventeen centuries.

nicipal Requirements. The laws of Boston require that wooden piles be capped with block-granite levelers or with Portland-cement concrete, that the spacing shall not exceed 3 ft between centers. The laws of Chicago are that wooden piles shall be driven to rock or hard-pan and capped with ge of timber, concrete, or steel, or a combination of these. The laws of York specify a minimum diameter of 5 inches and a maximum spacing of t between centers.

e Maximum Loads Allowed on Wooden Piles in various cities are as s: Atlanta, 20 tons; Philadelphia, 20 tons; Buffalo, 25 tons; Minneapolis, ns; Richmond, 25 tons; St. Louis, as many tons as the piles will safely t; Chicago, 25 tons; Louisville, 20 tons; St. Paul, 25 tons; New York, ns; Portland, Ore., 25 tons; Cleveland, 25 tons. Most of the above cities t the allowed load by Wellington's formula which is hereinafter given ge 193, under the heading, Bearing-Power of Piles.

nds of Wood Used for Piles. Wooden piles are made from the trunks es and should be as straight as possible and not less than 5 in in diameter e small end for light buildings, or 8 in for heavy buildings. The woods ally used for piles are spruce, hemlock, white pine, Norway pine, long-leaf short-leaf yellow pine, pitch-pine, cypress, Douglas fir, and occasionally oak, ry, elm, black gum and basswood. There does not appear to be much ence in the woods as to ility under water, but the er and stronger woods are preferred, especially where piles are to be driven to pan and heavily loaded.

eparing Wooden Piles Driving. The piles should REPARED FOR DRIVING by g off all limbs close to the t and sawing the ends e. It is probably better move the bark, although are more often driven with ark on, and it is doubtful if rk makes much difference ay or the other. For g in soft and silty soils, ence has shown that the drive better with a square t. When the penetration is han 6 in at each blow the f the pile should be pro- from BROOMING by put- ne IRON RING, about 1 in diameter than the head of e and from 2½ to 3 in y ¾ in thick. The head should be chamfered to fit the ring. When into compact soil, such as sand, gravel, or stiff clay, the point of e should be SHOD with iron or steel. The method shown at *A*, Fig. 31, s very well for all but very hard soils, and for these a CAST CONICAL POINT 5 in in diameter, secured by a long DOWEL, with a RING around the end pile, as shown at *B*, makes the best shoe. Piles that are to be driven in or

Fig. 31. Points of Wooden Piles Prepared for Driving

exposed to salt water should be thoroughly impregnated with creosote, d
or coal-tar, or some mineral poison to protect them from the TEREDO o
WORM, which will completely honeycomb an ordinary pile in three or fou

Driving Wooden Piles with the Drop-Hammer. The piles should
be driven to an even bearing, which is determined by the PENETRATIO
the last four or five blows of the hammer. The usual method of drivi
for the support of buildings is by a succession of blows given with a b
cast iron or steel, called the HAMMER, which slides up and down betw
uprights of a machine called a PILE-DRIVER. The machine is placed o
pile, so that the hammer descends fairly on its head, the piles alway
driven with the small end down. The hammer is generally raised by stean
and is dropped either automatically or by hand. The usual weight
hammers used for driving piles for building foundations is from 1 500 to 2
and the fall varies from 5 to 20 ft, the last blows being given with a sh
Occasionally, hammers weighing up to 4 000 pounds and over are used.

Driving Wooden Piles with the Steam-Hammer. Steam-ha
are to a considerable extent taking the place of the ordinary drop-ham
large cities, as they will drive many more piles in a day, and with less
to the piles. The steam-hammer delivers short, quick blows, from s
seventy to the minute, and seems to jar the piles down, the short inte
tween the blows not giving time for the soil to settle around them.† I
ing piles care should be taken to keep them plumb, and when the pen
becomes small, the fall should be reduced to about 5 ft, the blows being
rapid succession. Whenever a pile refuses to sink under several blows
reaching the average depth, it should be cut off and another pile driven b
When several piles have been driven to a depth of 20 ft or more and r
sink more than ½ in under five blows of a 1 200-pound hammer falling
is useless to try them further, as the additional blows only result in b
and crushing the heads and points of the piles, and splitting and crus
intermediate portions to an unknown extent.

Spacing Wooden Piles. Piles should be spaced not less than 2 ft n
than 3 ft, on centers, unless iron, wooden, or reinforced-concrete gr
used. When long piles are driven closer than 2 ft on centers there is dan
they may force each other up from their solid bed on the bearing s
Driving the piles close together also breaks up the ground and dimini
bearing power. When three rows of piles are used the most satisfacto
ing is 2 ft 6 in on centers across the trench and 3 ft on centers longitu
provided this number of piles will carry the weight of the building.
will not, then the piles must be spaced closer together longitudinally, or
row of piles driven; but in no case should the piles be less than 2 ft
centers, unless driven by means of a water-jet. The number of piles u
different portions of the building should be proportioned to the weigh
they are to support, so that each pile will receive very nearly the same l

Capping Wooden Piles. The tops of the piles should invariably
off at or a little below low water-mark, otherwise they will soon comm
decay. They should then be capped, either with large stone blocks, or
or with timber or steel grillage.

Granite Capping. Wooden piles are sometimes capped with bloc
LEVELERS which rest directly on the tops of the piles. If the stone do

* See Table XI, page 204.

† The 5 000 piles, averaging 48 ft in net length, under the Chicago Post O
driven with a steam-hammer weighing 4 400 lb and delivering 60 blows per mi

...of the pile, or a pile is a little low, it is wedged up with oak or stone ... In capping with stone a section of the foundation should be laid out ... drawings showing the arrangement of the capping stones. A single ... may rest on one, two, or three, but not on four piles, nor on three piles in ... line, as in the two last-mentioned ... it is practically impossible to make the ... bear evenly. Fig. 32 shows the best ... of the capping for three rows of ... Under dwellings and light buildings the ... often driven in two rows, STAGGERED, in ... case each stone should rest on three piles. ... the piles are capped, large footing stones, ... in single pieces across the wall, should ... in cement mortar on the capping. Fig. ... a partial piling-plan, with the arrange... of the capping stones, of the Boston ... of Commerce Building. It may be ... most of the stones rest on three piles, ... few on two piles.

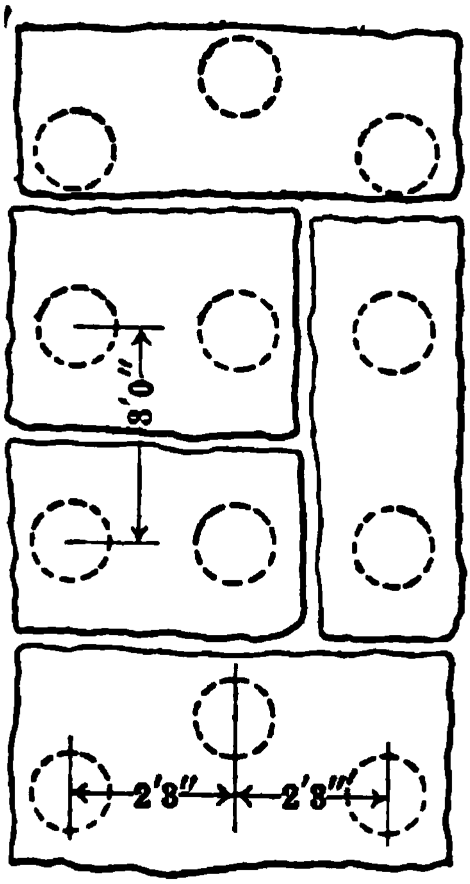


Fig. 32. Stone Capping for Three Rows of Wooden Piles

... Capping. In many buildings a very ... method of capping is to excavate to a ... of 1 ft below the tops of the piles and 1 ft ... of them and to fill the space thus ... solid with Portland-cement concrete, ... in layers and well rammed. After the ... is brought level with the tops of the ... additional layers are deposited over the ... width of the foundation until the concrete ... a depth of 18 in above the piles. On this foundation brick or stone ... are laid as on solid earth. If long bars of twisted steel, about ¾ in ... in cross-section are embedded in the concrete about 3 in above the ... the piles, the construction makes, in the opinion of the author, the best ... capping, the twisted bars giving great transverse strength to the concrete.

...-Grillage Capping. The pile foundations of many buildings have ... timber grillages bolted to the tops of the piles and stone or concrete foot... on top of the grillages. The timbers for the grillages should be at least ... in in cross-section, and should have sufficient transverse strength to ... the load from center to center of piles, using a low fiber-stress. They ... laid longitudinally on top of the piles and fastened to them by means ... -BOLTS, which are plain bars of iron, either round or square in section, ... into holes about 20% smaller in section than the bolts themselves ... or square bars 1 in in section are generally used, the holes being bored ... auger for the round bolts and by a ⅞-in auger for the square bolts. ... should enter the piles at least 1 ft. If heavy stone or concrete footings ... and the space between the piles and timbers is filled with concrete level ... tops of the timbers, no more timbering is required; but if the footings ... of small stones and no concrete is used, a solid floor of cross-timbers, ... 6 in thick for heavy buildings, should be laid on top of the longitudinal ... and drift-bolted to them. Where timber grillage is used it should, of ... be kept entirely below the lowest recorded water-line, as otherwise it ... and allow the building to settle. It has been proved conclusively, ... that any kind of sound timber will last practically forever if completely ... in water.

The Advantages of Timber Grillage are that it is easily laid and effe holds the tops of the piles in place. It also tends to distribute the p evenly over the piles, as the transverse strength of the timber will help t

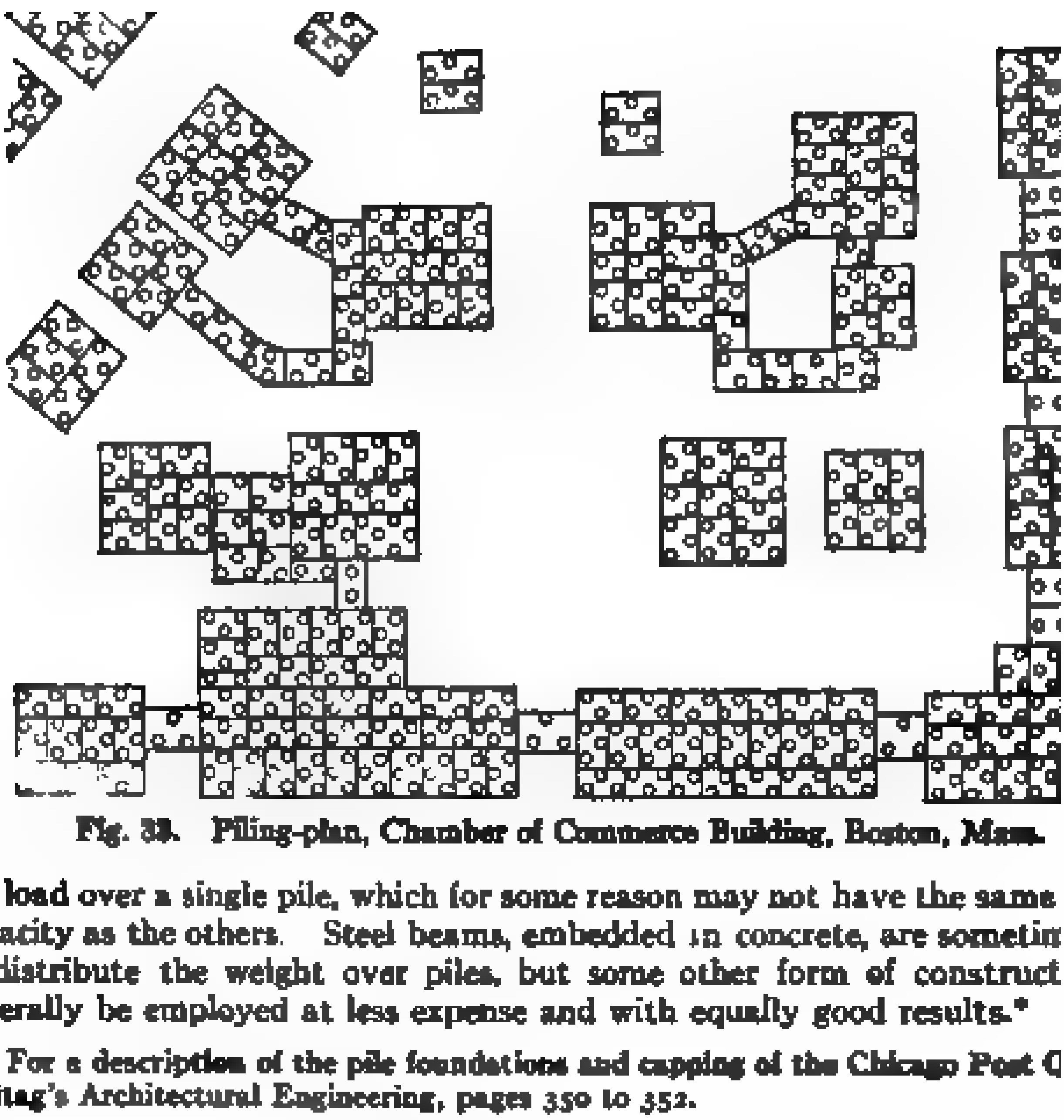


Fig. 39. Piling-plan, Chamber of Commerce Building, Boston, Mass.

the load over a single pile, which for some reason may not have the same capacity as the others. Steel beams, embedded in concrete, are sometim to distribute the weight over piles, but some other form of construct generally be employed at less expense and with equally good results.*

* For a description of the pile foundations and capping of the Chicago Post O Freitag's Architectural Engineering, pages 350 to 352.

...ifications for Wooden-Pile Foundations. This contractor is to fur... ...d drive the piles indicated on sheet No. 1.

...piles are to be of sound spruce (hemlock, long-leaf yellow pine) perfectly ...ht from end to end, trimmed close, and cut off square to the axis at both ...

...y are to be not less than 6 in in diameter at the small end, 10 in at the ...end, when cut off, and of sufficient length to reach solid bottom, the neces... ...length of piles to be determined by driving test-piles in different parts of ...undation.

...piles are to be driven vertically, in the exact positions shown by the plan, ...they do not move more than 5 in under the last five blows of a hammer ...ing 2000 lb and falling 20 ft. All split or shattered piles are to be re... ...d if possible and a good one driven in place of each imperfect one. In ...where such piles cannot be removed an additional pile is to be driven for ...imperfect one. If the piles show a tendency to BROOM, they are to be ...d with wrought-iron rings, 2½ in wide and ½ in thick.

...piles, when driven to the required depth, are to be sawed off square for a ...ental bearing at the grade indicated on the drawings.

...Bearing Power of Piles. In regard to their use for supporting build... ...piles may be divided into two classes: (1) Those which are driven to ...or HARD-PAN, that is, firm GRAVEL or CLAY and (2) those which do not ...HARD-PAN.

...A pile belonging to this class when driven through a soil that is sufficiently ...to brace the pile at every point, may be computed to sustain a load equal ...safe resistance to crushing on the least cross-section. If the surrounding ...plastic the bearing power of the pile will be its safe load computed as a ...n, having a length equal to the length of the pile when capped. Test... ...driven on the site of the Chicago Public Library Building, through 27 ft ...l, plastic clay, 23 ft of tough, compact clay and 2 ft into hard-pan, sus... ...d a load of 50.7 tons per pile for two weeks without apparent settlement. ...are many instances where piles driven to the depth of 20 ft in hard clay ...n from 20 to 40 tons, and a few instances where they sustain up to 80 tons ...e.

...A pile belonging to this class depends for its bearing power upon the ...ss, COHESION and BUOYANCY of the soil into which it is driven. The ...load for such piles is usually determined by the average penetration of ...e under the last four or five blows of the hammer. Several engineers ...formulated rules for determining the safe loads for piles of this class, but ...are so many conditions that modify the amount of the penetration, and ...act determination, and so many varying conditions of driving and of ...that it is considered impossible to formulate any rule that can be considered ...y satisfactory for all the conditions under which such piles are driven.

...Engineering News Formula. The formula generally used by engineers ...rived by M. A. Wellington, and is often referred to as the ENGINEERING ...FORMULA:

$$\text{The safe load in tons} = 2\,wh/\,(S+1)$$

...ich

w = the weight of the hammer in tons;
h = the height of fall of the hammer in feet;
S = the penetration in inches under the last blow or the average under the last five blows.

...loads are based on this formula the piles should be driven until the pene... ...does not exceed the limit assumed, or if this is found to be impracticable,

new calculations must be made based on the smallest average penetrati
can be obtained, and a greater number of piles used. In localities wher
is commonly used for foundations, the least penetration that can be ol
within practical limits of length of pile can generally be ascertained by o
tion, or by consulting somebody who is experienced in driving piles. The
the pile the less, as a rule, will be the final SET or penetration. Where the
experience to guide one it will be necessary to drive a few piles to det
the length of pile required, or the least SET for a given length of pile.
piles will have to be driven further than others to bring them to bear
equal resistance. When the piles are to be loaded to more than 50%
assumed safe load, the final set of each pile should be carefully measurec
inspector, the BROOM and SPLINTERS being removed from the head of t
for the last blow.

Safe Loads for Piles. Table X, computed by the above formula, gi
safe loads for different penetrations, under different falls of a hammer w
1 ton. FOR A HAMMER OF DIFFERENT WEIGHT multiply the safe load in tl
by the actual weight of the hammer in tons. Thus, for a hammer w
1 000 lb, the values in the table should be multiplied by ½ and for a :
hammer, by ¾.

Table X. Safe Loads in Tons for Piles

For hammer weighing 1 ton

Penetration of pile in inches	Height of the fall of the hammer in feet											
	3	4	5	6	8	10	12	14	16	18	20	25
0.25	4.8	(	1	9.7	12.9	16.1	19.4	22.5	25.8	29.1	32.3	..
0.50	4.0	5	7	8.0	10.7	13.3	16.1	18.7	21.3	24.0	26.6	33.
0.75	3.4	4	.7	6.9	9.2	11.5	13.8	16.1	18.4	20.7	23.0	28.
1.00	3.0	4	0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	25
1.25	.	3	5	5.4	7.1	8.9	10.7	12.5	14.3	16.1	17.9	22
1.50	..	3	0	4.8	6.4	8.0	9.6	11.2	12.8	14.4	16.0	20
1.75	.		6	4.4	5.8	7.3	8.8	10.2	11.7	13.1	14.6	18
2.00	..	.	3	4.0	5.3	6.7	8.0	9.3	10.7	12.0	13.3	16
2.50	..		.	3.4	4.6	5.7	6.9	8.0	9.1	10.3	11.4	14
3.00	..	.	.	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	12
3.50	.	.	.	..	3.6	4.4	5.3	6.2	7.1	8.0	8.9	11
4.00	..	.	.	..	3.2	4.0	4.8	5.6	6.4	7.2	8.0	10
5.00	.	.	.	..	...	3.3	4.0	4.7	5.3	6.0	6.7	8
6.00	..	.	.	.	...	...	3.4	4.0	4.6	5.1	5.7	7

Example of Computations for Pile Foundation. Suppose that fr
observations of the pile-driving for an adjacent building it is found tl
driven from 20 to 30 ft take a set of 1 in under a 1 200-lb hammer fallin
and that additional blows result in about the same set.

From Table X we find that the safe load for a fall of 20 ft and a p
tion of 1 in is 20 tons. Multiplying by the weight of the hammer in t
we have 12 tons as the safe load per pile. Suppose that the total lo
lin ft of footing is 13 tons. As we must have at least two rows of pi
each two piles will support 24 tons, it follows that the spacing of t
longitudinally should be 24/13 = 1 ft 10 in. As this is too close, we sh
three rows of piles, spaced 2 ft apart laterally, and the longitudinal

then be 36/13 = 2 ft 9 in. The width of the capping would be about If the load on the piles under the interior columns, for example, is 105.8 this divided by 12, the safe load for one pile, gives nine piles, or three rows he piles each, which should be spaced 2 ft 6 in apart, each way.

Actual Loads on Wooden Piles. The following examples of the loads supported by piles, under well-known buildings, and of loads which have borne for a short time without settlement, should be of value when ing pile foundations.

...ON. At the Southern Railroad Station three piles were loaded with 60 tons of pig iron, 20 tons per pile, without settlement. The allowed was 10 tons per pile.

...s 12 in in diameter at the butt and 6 in at the point, driven 31 ft into hard, clay near Haymarket Square, failed to show movement under 30 tons, ultimate load being probably 60 tons.* Other piles driven 17.9 ft sustained of 31 tons each. The average penetration under the last ten blows of a lb hammer falling from 9 to 12 ft varied from 0.4 to 0.95 in per blow for piles.

...s 25 ft long under the Chamber of Commerce Building penetrated about under the last blow of a 2 000-lb hammer falling about 15 ft.

...CAGO. In the Public Library Building the piles were proportioned to ns each and were tested to 50.7 tons without settlement.

...the Schiller Building the estimated load was 55 tons per pile; the building d from 1½ to 2¼ in.

...the Passenger Station of the Northern Pacific Railroad, at Harrison Street, 50 ft long were designed to carry 25 tons each and did so without per-...le settlement.

...e Art Institute Building, parts of the Stock Exchange Building and also e number of warehouses and other buildings on the banks of the river n piles.

...W YORK CITY. The Ivins (Park Row) Building is supported by about 14-in spruce piles, arranged in clusters of fifty or sixty, for single columns, a corresponding number under piers supporting two or more columns. ...les were driven to refusal of 1 in under a 20-ft fall of a 2 000-lb hammer. ...terial is fine, dense sand to a depth of over 90 ft. But few piles could be t more than 15 or 20 ft. The average maximum load per pile is 9 tons.† ...e American Tract Society's Building is supported on piles.

...OKLYN. Piles under the Government Graving Dock, driven 32 ft, on ...rage, into fine sand mixed with fine mica and a little vegetable loam, are ...ed to sustain from 10 to 15 tons each.

...W ORLEANS. Piles driven from 25 to 40 ft into a soft alluvial soil carry from 15 to 25 tons, with a factor of safety of from 6 to 8.‡

Cost of Driving Wooden Piles.§ The cost of driving piles naturally varies ...he character of the soil, and the conditions under which they are driven.

...W YORK CITY. A 2 500-lb drop-hammer drove 4 piles per day of 10 hours. ...steam-hammer, 13 piles per day were driven, for the same foundation. ...les were 70 ft long, 8 in in diam at the point and 15 in at the head. ...average cost of driving 800 piles with the steam-hammer was $2 each. In ...ark Harbor 1 800 piles were driven by a steam-hammer, from 24 to 26 ft ...vel and hard-pan, at a cost of 80 cts each.

...J. Howe, American Architect, June 11, 1898.

...a description of this foundation, see the Engineering Record of July 23, 1898.

...L. Patton.

...e prices are now (1920) considerably higher.

CHICAGO. Forty Norway-pine piles were driven by a firm of contr 15 ft deep every ten hours at a cost, for driving, of 55 cts each. Anoth drove from 60 to 65 piles, each 45 ft long and 15 ft deep, into hard sand ea at a cost of about 30 cts each. In both cases steam-hammers were used.*

BOSTON. Spruce piles from 30 to 45 ft long cost from \$3 to \$5, in Long-leaf yellow pine piles, as long as 70 ft, cost about \$15 apiece for th themselves, and \$2 or more each for the driving. Oak piles from 40 to long cost from \$8 to \$10 each, in place.†

Some Other References to Wooden Piles and Pile-Driving. A valuable paper on "Some Instances of Piles and Pile-Driving, New and by Horace J. Howe, was published in the American Architect and Building commencing June 11, 1898. The paper records a great many tests and several formulas and many experiences of distinguished engineers. Pa Building Construction and Superintendence, by F. E. Kidder, gives add information in regard to pile foundations and experiments on the bearing of piles. Much valuable information on piles is given in "A Practical T on Foundations," by W. M. Patton. The recent Engineers' Handbook should be consulted for additional data.

28. Concrete-Pile Foundations

Durability of Wooden and Concrete Piles. Concrete piles, either or reinforced, possess many advantages over wooden piles and, in gener be used in all places where wooden piles can be driven. Concrete piles pared with wooden piles, have primarily the advantage of greater PERMA Timber piles, kept constantly wet and protected from the action of the or other destructive influences, may be practically everlasting, but can counted upon above water level; whereas concrete piles should be proof all deteriorating actions, whether wet or dry, except the action of free wet concrete.

Strength of Wooden and Concrete Piles. Concrete piles withou forcement, if made of good concrete, should have nearly the same CR STRENGTH per square inch as ordinary yellow-pine piles, and with p placed reinforcement concrete piles should have a much higher crushing st per square inch than timber piles. Moreover, timber piles do not hav FORM CROSS-SECTIONS. For instance, a slender timber pile 40 ft in leng 12 in in diameter at the butt, is probably not over 6 in in diameter at the In direct compression the load on a point-bearing pile of the above dim is limited to the safe load on the point of the pile, where it is 6 in in dia and a cylindrical concrete pile, 12 in in diameter and under similar con will have a cross-section of 113 sq in at all points, compared with the section of 28 sq in at the point of the timber pile. Moreover, if we c both piles as LONG COLUMNS, it must be borne in mind that a timber pile n be straight and that it may, therefore, be subject to STRESSES and DEFOR due to ECCENTRIC LOADING, which are avoided in a straight, concrete pil

Reinforced-Concrete Piles. In practice concrete piles are general forced, and if a pile is to be considered as a long column the reinforceme be increased at the center, so as to provide for stresses due to handling its acting as a long column. The concrete piles may be formed co above ground, in which case they may be straight or tapered, with squ ular or other cross-sections. The reinforcement may consist of a nu

* American Architect, June 4, 1898, page 78.

† George B. Francis, in American Architect, July 23, 1898.

[...]al rods generally disposed symmetrically around the axis of the pile. The [...]al rods should be connected by horizontal wiring or by spiral reinforcement. [...]fore stated, the reinforcement may be increased at the central section so [...] provide against stresses due to the use of the pile as a LONG COLUMN, in [...] case the additional reinforcement should be placed near the periphery [...]e cross-section.

[...]ypes of Concrete-Pile Reinforcement. There are many TYPES OF REIN[...]CEMENT, one method even employing a woven-wire fabric which is laid out [...] a table and covered with a thin layer of concrete, the entire mat comprising [...]ire fabric and the concrete being then rolled into a solid cylindrical form [...], when set, forms the finished pile. The concrete piles may be FORMED IN [...]E by any one of several different methods.

[...]e Raymond System of Concrete Piling. In this system of concrete [...] a permanent form is provided for each pile. The Raymond system [...]sts of a collapsible steel mandrel or core, tapering from 8 in in diameter [...]e point at the rate of 0.4 in per foot in length, until in a length of 37 ft [...]diameter equals 23.2 in. Upon this expanded mandrel or core is placed [...]rally reinforced sheet-metal shell, the reinforcement of which is grooved [...] the metal on 3-in centers the entire length of the core or pile. This rein[...]ment imparts rigidity and stiffness to the shell, renders it capable of with[...]ing very severe soil-pressure and prevents admixture of foreign substances [...] the green concrete. The combined mandrel and shell is driven into the [...]d to a proper refusal; the mandrel is then collapsed and withdrawn from [...]hell, leaving the shell permanently in the ground; and the interior of the [...]er form is then inspected, and when perfect from tip to top, is filled with [...] concrete. Thus the pile is completed. The extreme taper of the shell, [...]ined with the friction between the shell and the surrounding soil, increases [...]arrying capacity of the pile. The safe load on a Raymond pile varies [...] 15 to 30 tons.

[...]e Simplex Method of Forming Concrete Piles in Place. THE SIMPLEX [...]OD differs from the Raymond method and may be briefly described as [...]s: A steel pipe, generally cylindrical in form, of the required size and [...] and fitted with a detachable cast-iron conical DRIVING-POINT, is driven into [...]round to the required depth; the pipe is then partially filled with concrete. [...]on-like PLUNGER, smaller in diameter than the inside diameter of the [...] is then placed on the concrete and the pipe is partially withdrawn, leaving [...]riving-point and part of the superimposed concrete in the ground. This [...]tion is repeated until the pile is built up to the required height. In cer[...] materials, instead of using a detachable driving-point, the driving-point [...]s of two jaws hinged to the lower end of the pipe, so arranged that while [...] the driving they form a driving-point, when the pipe is withdrawn they [...]nd form an extension of the cylindrical pipe. In other words, the jaws [...]med of steel plates previously bent to the same radius as the radius of [...]pe and so hinged that when they are in their open-position the plates [...]g the jaws constitute an extension of the cylindrical surface of the pipe. [...]vident that plain reinforcing-bars can be placed in position before concrete [...] into the pipe.

[...]on for Concrete Piles Built in Place. Care should be taken in design[...]d placing the reinforcing for all concrete piles BUILT IN PLACE, that the [...]ent placing of the concrete does not throw the reinforcement out of [...] and that all voids between the reinforcement and the shell are com[...]y filled.

The Pedestal Pile is designed to give an ENLARGED CROSS-SECTION a base of the pile. The method is similar to that of the RAYMOND METHOD increase in diameter being obtained as follows: After the pipe has been d the driving-core is withdrawn and the pipe partially filled with concrete. the concrete in the pipe is rammed, forcing the concrete out of the pipe and pressing the material below the pipe, so that the concrete is forced into th A repetition of this operation results in forming a BASE or MUSHROOM belo pipe larger in diameter than the diameter of the pipe. Finally the pipe is drawn, the filling and ramming-operations continuing meanwhile, until th is carried up to the required height.

Composite Piles. Protected piles, for use in localities where the TE affects the life of timber piles under water, are composed of timber piles concrete coatings held in position by steel reinforcements in the shape of exp metal or wire netting. Such piles are to be considered as timber piles than as concrete piles.

Timber Piles with Concrete Caps. In some localities where the perm water-level is considerably below the level of the required excavation, t piles have been driven with a FOLLOWER, the follower consisting of a stee or cylindrical shell. When the head of the pile is driven to a safe dis below low water the PIPE-FOLLOWER is filled with concrete and withd leaving the concrete pier resting on a timber pile. This composite pile appear to possess the advantage of combining the cheapness of a timbe below the water-level with the permanency of a concrete pile above the level. Great care, however, should be used in adopting this method on ac of the difficulty of securing proper connection between the concrete an wooden pile.

The Methods used in Driving Built-up Piles are practically the as are used in driving wooden piles, except that a CUSHION of wood, ro other material is placed on the head of the pile to be driven to cushion the of the hammer. Steam-driven or air-driven RECIPROCATING HAMMERS are erable to the ordinary DROP-HAMMERS. In stiff materials the use of a w JET is advisable and, in fact, in many cases indispensable. In lifting co piles use is made of a special SLING which is attached to a pile at two points point one-quarter of the length of the pile from the end. The sling should a SPREADER so that the stress due to the oblique pull of the CHAIN-SL taken up by the spreader rather than by the pile.

The Casting of Concrete Piles. Concrete piles should be CAST I PIECE by a continuous operation so that there will be no PLANE OF WEA formed between partially set concrete and fresh concrete. They may b either in a vertical position, in forms, or in a horizontal position. Squa tion concrete piles have been cast in a horizontal position and side-forms used, the previously cast concrete pile, protected by paper, forming the b form. In some cases, where it is intended to use a WATER-JET in sinking the latter is cast around an iron pipe which is afterwards used for the jet. In general, however, this is dispensed with and an external deta pipe used for the water-jet.

Incidental Advantages of Concrete Piles. In many cases, where co piles are more expensive than timber piles, the saving in excavation and ings more than offsets the increased cost. For example, if the excavati the cellar of a building does not go down to water-level, the use of timbe will necessitate excavating down to a point below water-level in order th piles may be cut off low enough to keep their heads always wet. Co

s, however, can be driven from the level of the bottom of the cellar-excava- , and this additional excavation and the necessary construction between excavation-level and the level of the cut-off for the timber piles thus avoided. ever, as one concrete pile may have a SUPPORTING POWER equal to the porting power of four wooden piles, the size of the footings will be much ller with concrete piles than with wooden piles.

Comparison of Wooden and Concrete Piles under Piers. The footings a column or pier 24 in sq in section, requiring for its support, say, sixteen oden piles, spaced 2 ft 6 in from center to center, will be, allowing for slight ualities in driving, approximately 10 ft square, the projections being 4 ft nd the size of the base. Such a footing will ordinarily require a steel ge or reinforced-concrete base, or, if made of ordinary concrete, will be of y considerable depth; whereas, if four concrete piles, placed 3 ft from center enter, are used, instead of wooden piles, the area of the base will be a little r 4 ft square and the projection will be only 1 ft. A suitable footing would sist of a reinforced-concrete cap not over 2 ft in thickness. The saving in l of excavation, concrete and steel in the footing is all in favor of the use oncrete piles.

Concrete Piles under Walls. In the case of a continuous wall, where the d per linear foot of wall is not great, a single row of concrete piles is often cient to support the weight of the wall. In such cases, the piles should be placed in straight lines but should be STAGGERED, and a sufficient footing ld be constructed connecting the heads of the piles, so as to afford stability he wall.

The Method Employed in Calculating Reinforcement for Concrete s is the same as that employed in calculating ordinary reinforced-concrete mns, the only difference being that where a pile is not point-bearing, but is ndent on the surrounding material for its support, it need not be considered LONG COLUMN. POINT-BEARING PILES deriving their support from some material on which their lower extremity rests, must be considered as LONG mns, on the assumption that the material surrounding the piles may fail upport them. In the case of FRICTION-PILES, depending for their support the surrounding material, this assumption cannot be made, as any failure e material will involve a settlement of the pile. It should be borne in mind t any structure supported on piles supported by SKIN-FRICTION is dependent its stability upon the continued supporting power of the material surround- the piles. In many cases buildings resting on piles driven into soft ground e settled as the result of the consolidation and settlement of the material unding the piles, notwithstanding the fact that the piles when driven were y able to support the loads for which they were designed.

n-Pipe Piles with Concrete or Reinforced-Concrete Filling have been d in place of wooden or concrete piles, especially in UNDERPINNING-WORK. objection to the use of such piles is that the iron pipe forming the external may rust, in which case the strength of the pile is reduced to the strength e concrete filling and the reinforcement contained therein. The writer ves that they should not be used for permanent work.

ds Allowed on Concrete Piles. The building laws of most cities on concrete piles from 350 to 500 lb per sq in on the concrete plus from to 7 500 lb per sq in on the vertical reinforcement. On this statement ld appear possible to design a short concrete pile 12 in square, on which owed load would be 100 tons, and it is possible that such a pile, tested SHORT COLUMN, would develop in a testing-machine a strength justifying

the use of such construction; but, bearing in mind that the character o support for the base of such a column is underground and cannot be insp and bearing in mind also the uncertainties attending the manufacture o pile, it is evident that it would be improper to load a pile to this extent in pra It would, however, be considered good practice to load concrete piles up tc third of a test-load applied to not less than 3% of the piles used. In ord practice, reinforced-concrete piles are loaded up to 500 lb per sq in of cros tion.

29. Foundation Piers and Foundation Walls

Foundation Piers and Walls as distinguished from ordinary CELLAR and WALLS, extend from the level of the underside of the cellar-floor to ro other solid foundation-bed. (See page 129, Subdivision 1, and also Ch III, pages 228–9.) In general, such piers and walls are composed of co and are of such dimensions that the safe unit loads on the concrete fo them are not exceeded. If the foundation-bed is rock, compact hard-pa gravel, there need be little or no enlargement of the base of the pier or wa the safe unit loads on such natural foundation-beds are generally equal t safe unit loads on the concrete forming the body of the pier or wall. The o of such piers and walls is therefore an entirely simple matter governed b principles already outlined, and by certain considerations mentioned here

The Methods used in the Construction of Foundation Piers Walls are, however, necessarily varied to suit different materials and to different conditions encountered, and the design of a pier necessarily differ different methods of construction. For example, if the construction is executed by means of the ordinary SHEET-PILING METHOD, piers and wal have in general rectangular outlines. But if the CHICAGO METHOD or the MATIC CAISSON is employed, it will generally be cheaper to use piers ha circular cross-section and the support for walls may be a succession of cyl rather than continuous walls. The detailing of the concrete structure c tuting the piers or walls is simple after a determination is made of the me by which the construction is to be put in place. This subject is discussed following chapter-subdivision, Methods of Excavating for Foundations.

30. Methods of Excavating for Foundations

Simple and Complex Excavations. Excavations in earth for footi walls and piers may vary from simple trenches and pits of the require and depths to accommodate the footings, up to deep subaqueous excav requiring all the resources of engineering skill.

The Sides of Excavations. If the earth is firm and the depth not ex the sides of the excavation may be self-supporting, in which case the exca may be made the neat size of the footing and the sides of the excavatio take the place of forms for the concrete deposited to form the footing. the excavation is deep, and especially where the earth is not firm, the s the excavation must be sloped or, if made vertical, must be supported by b or by some form of sheet-piling. Where the excavation is over 8 ft in d will generally be cheaper to support the sides of the excavation than t them. Where the excavation adjoins a property-line it will generally b visable to slope the excavation on account of damage to the adjoining pr and in such cases it will be necessary to use sheeting, even if sloping th would be cheaper.

bracing in many cases will serve to support the sides of the excavation without the necessity of close SHEETING. The BRACING may consist simply of short pieces of PLANK placed against opposite sides of the excavation and held in position by horizontal timber STRUTS secured by WEDGES; or, especially in narrow trenches, some form of EXTENSIBLE SEWER-BRACE may be used. Fig. 34 represents a usual form of EXTENSIBLE BRACE. Generally, however, the sides of an excavation will not stand with a vertical face, even if braced in this manner, for any length of time, and if the material is sand or soft clay, such bracing is entirely inadequate. In such cases, and in fact generally, some form of CONTINUOUS SHEET-PILING must be employed.

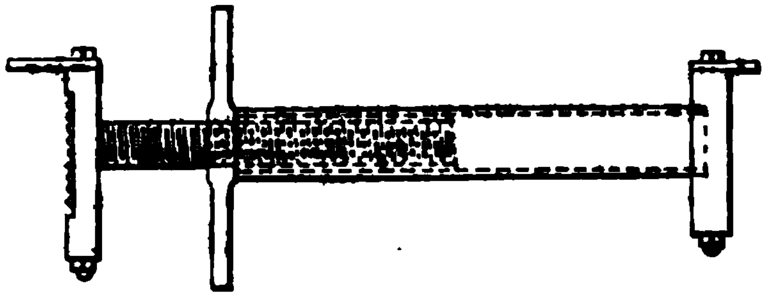


Fig. 34. Extensible Brace for Narrow Excavations

Ordinary Wooden Sheet-Piling consists of a continuous line of vertical planks held against the sides of the excavation by horizontal timbers known as WALES, WALING OR BREAST-TIMBERS, these wales, or breast-timbers being in turn supported either by CROSS-BRACES extending across the excavation to an opposite wall or side of the excavation, or by inclined struts known as SHORES or RAKERS, extending to the bottom of the excavation where HEELS or inclined platforms are sunk in the undisturbed material to afford points of support.

Earth-Pressure on Sheet-Piling. The load on the sheeting due to the EARTH-PRESSURE may be calculated on the assumptions made for the design of RETAINING-WALLS, but the thickness of the sheeting planks, the sizes and spacing of the breast-pieces and braces, if figured on this basis, will in general exceed the sizes constantly used with success and safety in such work. The probable reason for this is that an earth bank, when steadied and in part supported by the sheeting, does not, for a considerable time, lose the COHESION between its particles natural to most earth banks in their original and undisturbed state. Or, in other words, under these conditions no real ANGLE OF FRICTION is developed in the earth-mass. Local experience and practice should be consulted and will usually serve as a guide. Earth banks apparently similar will, however, act very differently and no general rule can be given. It should be borne in mind that the earth composing a bank should be, as far as possible, protected from drying, from the action of water and from alternating freezing and thawing; and that permanent work should be completed as rapidly as possible so as to avoid the deteriorating effects of time and exposure on the structure of the bank.

The Thickness of the Sheeting Planks required may be calculated on the assumption that the earth bank is composed of loose material having a definite ANGLE OF SLOPE and COEFFICIENT OF FRICTION; but practically, under favorable conditions, 2-in planks may be used for a depth of drive of 16 ft, 3-in planks up to 24 ft and 4-in planks up to 32 ft; and timbers, 8 by 12 in, have been driven in suitable material to a depth of over 40 ft.

Depths and Numbers of Drives. Ordinarily the depth to which a plank can be driven is limited by its ability to resist the shock due to driving, and in unsuitable material a plank may become shattered before it is driven to the above-quoted depths. If the required depth cannot be reached by the first planks driven, a second, and sometimes a third and fourth set of planks are employed. The BREAST-PIECES supporting the first line of planks must remain in place,

the planks in the second set or DRIVE have to be placed inside of the b
pieces, thus reducing the size of the excavation by the amount of the nece
offset. Where more than one drive is required the first drive should be st
at a sufficient distance outside to allow the planks forming the second o
second and third drives to be placed outside of the required area for the
tom of the excavation.

Cutting and Fitting Sheeting Planks. The sheeting planks ma
SQUARE-EDGED where there is no water or fine loose sand, but where
or running sand is to be excluded the planks should be TONGUED
GROOVED, or SPLINED. The use of tongued and grooved planks ha
additional advantage that the planks are more readily kept in line.

Fig. 35. Small Power-hammer for Driving Sheeting Planks

Fig. 36. Large-size Power-hammer and Sheeting Planks

usual to cut the bottom edge of each plank on a slight angle, so that in dr
it is WEDGED against the preceding plank. The top of each plank may be f
to receive an iron DRIVING-CAP; or, if this is not used, the upper corners o
plank should be cut off so that the effect of the blows will be concentr
along its vertical axis, and the tendency of the plank to SPLIT, due to a
on one corner, thus diminished.

The Means Employed for Driving the Sheeting vary with the d
and the size of the sheeting. For small jobs and for moderate depth
drive, the primitive method of DRIVING BY HAND with ringed wooden m
still prevails. For work involving a considerable amount of driving, and i
cases for long drives, POWER-HAMMERS driven either by steam or compresse
are preferably employed. A small-sized power-hammer (Fig. 35) resemb
STEAM-DRILL and may be handled by two or three men without any special lif

...tion. The larger sizes of power-hammers (Figs. 36 and 37) are practically ..., power, pile-driving hammers arranged with a special DRIVING-HEAD to ... the sheeting employed. Such hammers are handled by DERRICKS or are ...ied in a frame similar to a pile-driver frame. Ordinary DROP-HAMMERS ...sometimes used, but are not as advantageous as the RECIPROCATING POWER-...MERS, as the blow struck by the drop-hammer ...ures the plank, while the frequent light blows ...the power-hammer tend to keep the planks and ...adjacent material in motion and accomplish the ...quired work with less damage to the sheet-piling. ...weights and dimensions of several types of pile-...ving hammers are given in Table XI, page 204.

...anner of Driving Sheeting Piles. In prac-...ice, a shallow excavation is first made to the ...per line for the outside of the sheeting planks. ...he top BREAST-TIMBER is temporarily secured in ...ace and the lower end of the planks placed be-...ween this timber and the bank. If the planks ...re long, temporary TOP GUIDES or STAY-BRACES are ...ranged so as to keep the planks vertical until ...ey have been driven well into the ground and ...ured by the permanent BREAST-PIECES. The ...anks are then driven as the excavation pro-...resses, each plank being driven a few inches in ...rn. As the driving goes on the material under ...e lower edge of the planks is loosened with a ...ovel or with a crowbar, the operation being so ...ducted that the planks are held true to line. ...he horizontal breast timbers and their braces are ...aced in position as the excavation progresses. ...inclined braces are to be used the excavation in ...e center is taken out first, leaving a sloping bank ...ainst the sides of the excavation. This permits ...the placing of the inclined braces and of the ...ds for their points of support before there is any ...nger to the bank. After the first breast piece ...d its inclined brace are set in place, the second ...d subsequent breast-pieces and braces are put in ...the excavation proceeds.

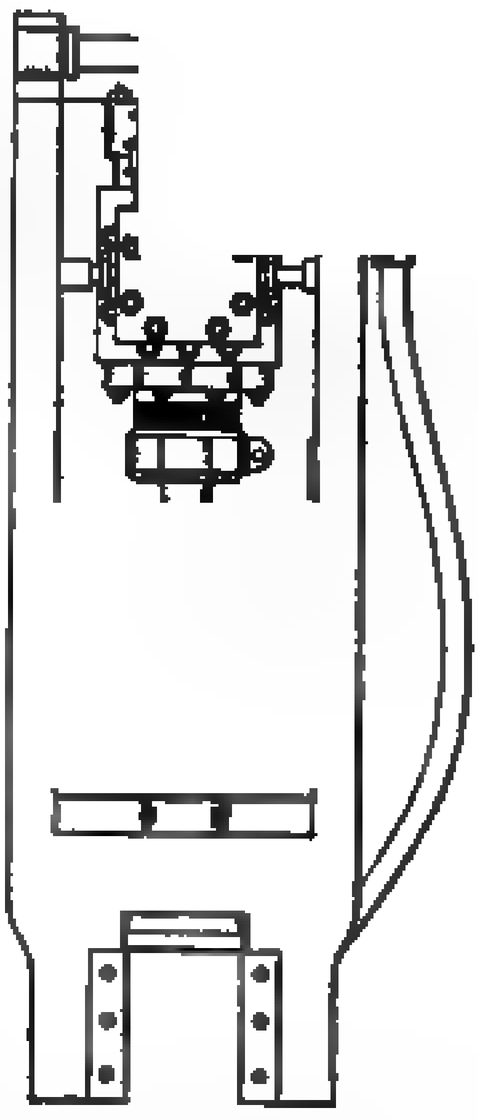


Fig. 37. Large-size Power-hammer for Driving Sheeting Planks

...eet-Piling for Excavations Below Water-...el. These excavations may be made by ...SHEET-PILING METHOD if there is not too much water and if water can ...ained out of the material without inducing a flow of sand or clay below ...bottom of the sheet-piling. In some cases, where unfavorable conditions ...t, but where there is an underlying stratum of impervious material, it is ...ible to drive the sheeting in advance of the excavation, so that the bottom ...e sheeting makes a tight joint with the impervious stratum, cutting off the ...of water and material. Where a considerable amount of water finds its ...into the excavation, the water must be led to a SUMP or depression from ...it is ejected by means of a PUMP or a STEAM-SYPHON. Where the founda-...d is below water-level and the material is sand, clay, or other material ...would be softened by the action of the water, it should be protected by ...g the sump at a considerable distance from the area to be used for the sup-

Table XI. Weights and Dimensions of Pile-Driving Hammers

Size-No.	Total net weight	Weight of ram	Dimensions over all			Cylinder			Steam-boiler H. P. required	Comp. air, free air per min	Size of hose	Distance between jaws	Width of jaws	Duty, size of … or piling hammer will dri…
			Height	Width	Depth	Diam.	Stroke	Strokes per min						
	lb	lb	in	in	in	in	in			cu ft	in	in	in	
Warrington Steam Pile-Hammers. Manufactured by Vulcan Iron Works, Chicago, Ill.														
0	16 000	7 500	180	....	....	16½	48	60	60	....	2½	26	9¼	Hvy concrete
1	9 850	5 000	144	....	....	13½	42	65	40	....	2	20	8¼	18″ sq or rd pi
2	6 500	3 000	138	....	....	10½	36	70	25	....	1½	19	7¼	14″ sq or rd pi
3	3 800	1 800	96	....	....	8	30	80	18	....	1¼	18	6¼	10″ sq or rd pi
4	1 350	550	84	....	....	4	24	80	8	....	1	14	4¼	4″×12″ sheet
5	800	....	68	10	10	4	7½	300	10	....	1	....	...	3″×12″ sheet
Cram Steam Pile-Hammers. Manufactured by A. F. Bartlett & Co., Saginaw, Mich.														
B	8 400	5 500	144	....	....	....	40	70	25	....	2½	27	8¼	18″ sq or rd p
C	5 500	3 090	144	....	....	....	40	70	18	....	2	20	8¼	14″ sq or rd p
D	4 200	2 250	102	....	....	....	24	80	15	....	1¾	20	8¼	10″ sq or rd p
E	1 000	430	78	....	....	....	12	80	15	....	1¼	12	5¼	4″×12″ sheet
Union Pile-Hammers. Manufactured by Union Iron Works, Hoboken, N. J.														
0	12 100	2 550	118	28	20	10½	24	100	50	750	2	28	8½	Hvy concrete
1	8 000	1 548	94	28	18	9½	21	110	30	600	1½	28	8½	18″ sq or rd p
2	5 500	890	81	25	15	7¼	16	130	18	300	1¼	25	6½	14″ sq or rd p
3	4 500	663	74	23	13	6¼	14	135	15	200	1¼	23	5½	10″ sq or rd p
4	2 500	363	60	20	11	5¼	12	150	10	150	1	20	4½	6″×12″ shee
5	1 400	214	47	17	9	4¼	9	200	8	100	1	17	4½	4″×12″ shee
6	850	129	40	14	8	3¼	7	250	5	60	¾	14	3⅜	2″×12″ shee
7	365	70	31	10	6	2⅜	5	300	3	40	¾	10	3⅜	1″×6″ sheeti
Goubert Steel-Pile Driving-Hammer. Manufactured by A. A. Goubert, New York, N. Y.														
3	5 000	1 500	76	29	17	8	14	150	50	660	2	24	8¼	18″ sq or rd p
2	3 400	800	62	24	14	6¼	10	160	25	340	1½	22	6¼	12″ sq or rd p
1	950	200	43	16	10½	4	8	200	10	150	1¼	....	...	4″ sheeting
New Monarch Steam Pile-Hammer. Manufactured by Henry J. McCoy Co., New York, N. Y.														
1	7 000	1 500	90	24	24	9	12½	125	35	600	2	24	8¼	18″ sq or rd p
2	4 600	850	72	20	20	7¼	11	150	20	300	1½	20	8¼	14″ sq or rd p
3	2 800	450	54	18	18	4¾	7	175	15	150	1	18	8¼	6″×12″ shee
4	800	125	48	14	14	3¼	6	250	10	65	¾	....	...	3″×12″ shee
McKiernan-Terry Pile-Hammers. Manufactured by McKiernan-Terry Drill Co., New York, N. Y.														
9	7 500	1 500	77	21	27⅜	15	12	200	60	600	2	21	6½	18″ sq or rd p
7	5 000	800	67	21	22⅜	12½	10	225	35	350	1½	21	6½	14″ sq or rd p
5	1 500	200	56	11	14⅞	7	8½	300	20	200	1¼	11	4¼	4″×12″ shee
3	640	68	54	9	9½	3¼	5¾	300	15	150	1	9	3½	3″×12″ shee
1	145	21	42	8	6½	2¼	3¾	500	10	100	¾	8	2⅛	2″×10″ shee
Ingersoll-Rand Sheet Pile-Driver. Manufactured by Ingersoll-Rand Co., New York, N. Y.														
G1	1 200	200	80	11¼	11	4	7¼	300	10	110	1¼	....	...	4″×12″ shee

of the footing. This may be accomplished by making the area to be sheeted excavated large enough to accommodate the sump outside of the supporting area, or by sinking a separate excavation to be used exclusively as a sump; the same result may in some cases be accomplished by the use of DRIVE-WELLS, driven to a point below the level of the footing in which continued pumping may reduce the level of the water to a point below the footing. Care should be taken, when the level for the footing is reached, to prevent the foundation-bed from being disturbed and softened by unnecessary tramping of workmen over the surface of the excavation.

The foundation-bed should be left as nearly as possible in its original or natural condition.

Steel Sheeting has been largely employed recently in place of wooden sheeting. It has the advantage that it can be driven in advance of the excavation, thereby reducing the likelihood of any flow of material under the sheeting. It also has the advantages of affording greater strength for a given thickness of sheeting, of being driven to a greater depth, and in many cases of being withdrawn and used over again. As generally manufactured, it has the further advantage of being INTERLOCKING, so that there is less danger of its getting out of line and leaving openings between adjacent pieces.

All of these advantages have been considered by engineers in using steel instead of wooden sheeting.

The Use of Steel Sheeting. The fundamental idea of steel sheeting is not new, as CAST-IRON SHEET-PILING was used in England as far back as 1822 and various combinations of steel plates have been used in coffer-dams. The general use of steel sheeting started in this country in 1899 when Luther P. Friestedt made experimental INTERLOCKING CHANNEL-BAR SECTIONS. Since that time it has come into general use, and with its aid many excavations have been made with STEEL SHEET-PILING which would have been impracticable with timber sheeting.

Earth-Pressure on Steel Sheeting. In using STEEL SHEETING, it should be borne in mind that the EARTH-PRESSURE coming on the steel sheeting is the same as the earth-pressure coming on timber sheeting, and the breast-pieces and braces should be as strong as in the case of timber sheeting. Certain forms of sections of steel sheeting offer considerable resistance to bending due to the lateral earth-pressure. With such forms the horizontal breast-pieces may be spaced farther apart than with ordinary timber sheeting or steel sheeting not having this property; but the strength of the breast-pieces and of their braces must be sufficient to take up the entire load coming on the sheeting, irrespective of the spacing between such breast-pieces, for in case there is a failure in these the entire sheeting will fail.

Different Forms of Steel Sheeting. Various TYPES OF STEEL SHEETING are on the market. In making a selection between different forms of sheeting, the character of the material to be encountered should be borne in mind, as the simpler, more compact sections will penetrate hard or gravelly soils with less danger of deformation than the more complicated sections made up of thin plates and shapes. The various companies manufacturing different forms of sheet-piling publish catalogues containing data as to the weight and also giving properties of the different sections. These catalogues may be obtained from the manufacturers, but for convenience illustrations of some of the principal sections, with their dimensions and weights and other details, are given in the following pages.

There are other types of steel sheeting than those shown in Figs. 38 to 44.

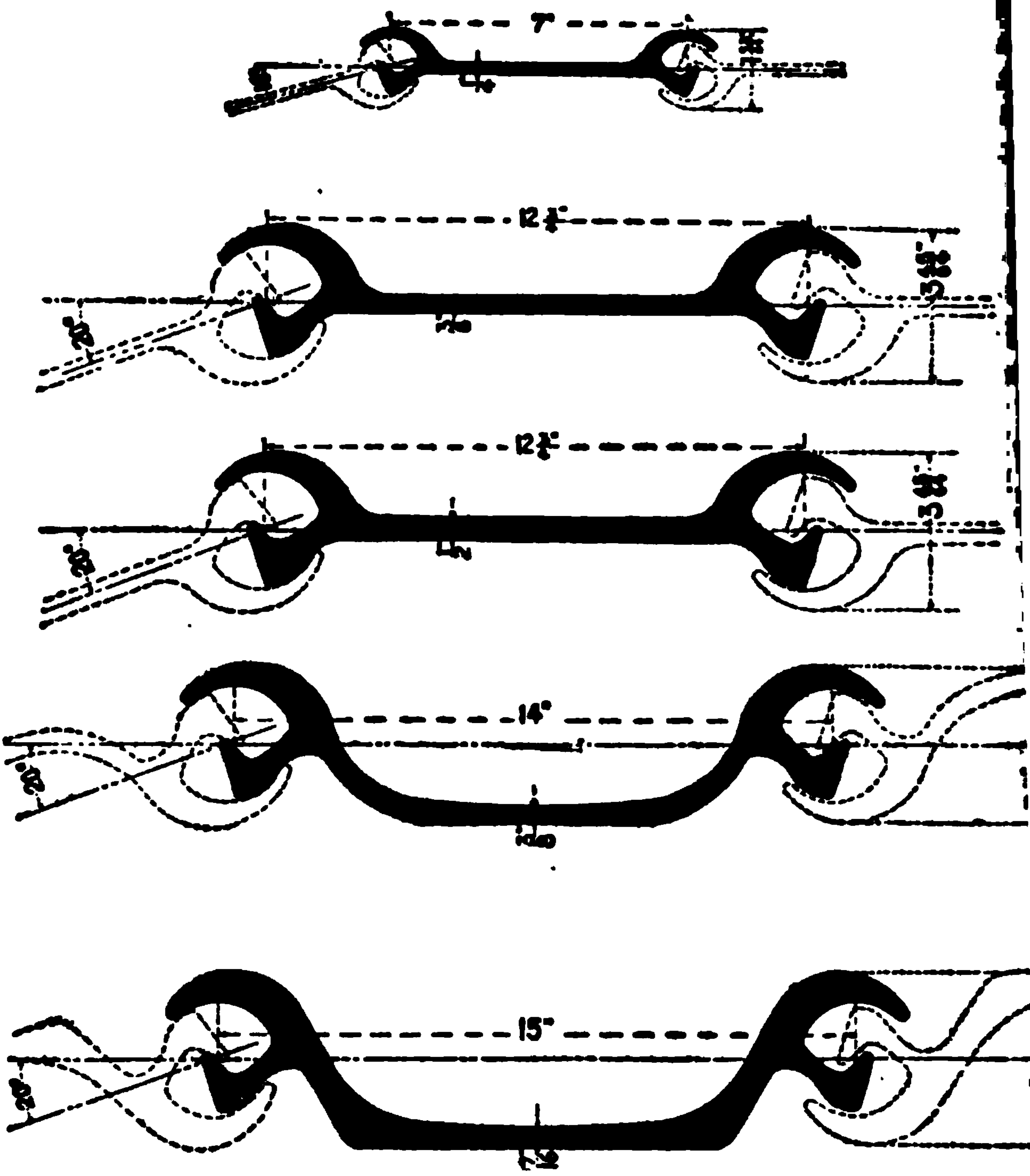


Fig. 38. Lackawanna Steel Sheet-piling

Lackawanna Steel Sheet-Piling *

Composition and Dimensions of Sections

Sections	Per linear foot, lb	Per square lb
Straight-web, ¼ in thick	12.54	21.5
Straight-web, ⅜ in thick	37.187	35
Straight-web, ½ in thick	42.5	40
Arched-web, 14 in long	40.83	35
Arched-web, 15 in long	60	48

This piling is adapted to straight or circular work.

* Manufactured by the Lackawanna Steel Company.

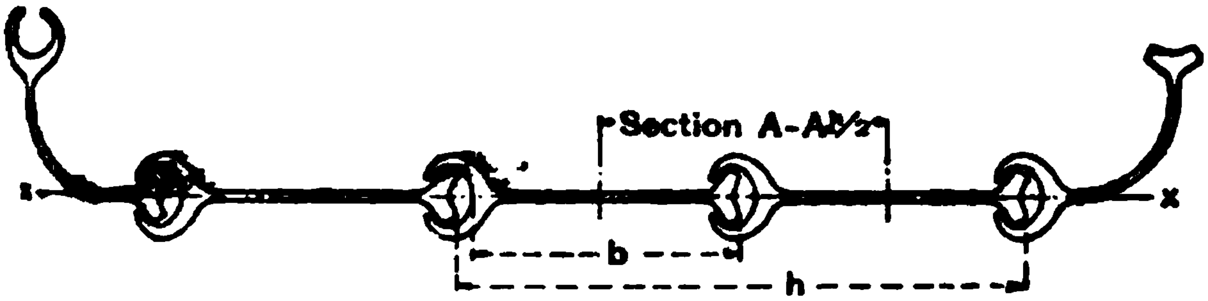


Fig. 39. United States Steel Sheet-piling

United States Steel Sheet-Piling *

Composition and Dimensions of Sections

Size	Web in	*b* in	*h*/2 in
12½ in, 38 lb..........	⅜	12½	13¼
9 in, 16 lb............	¼	9	9¼

This piling is adapted to straight or circular work.

* Manufactured by the Carnegie Steel Company.

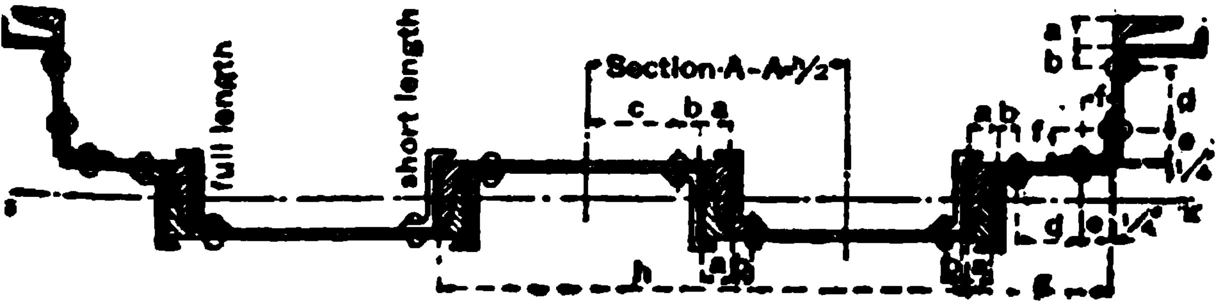


Fig. 40. Friestedt Interlocking Channel-bar Piling

Friestedt Interlocking Channel-Bar Piling *

Composition and Dimensions of Sections

No.	Description	Channels		Zees		*h*/2, in
		In	Lbs per ft	In	Lbs per ft	
1	10 in, 28 lb	10	15	3⅛×¼	4.8	9
2	10 in, 34 lb	10	20	3⅛×¼	4.8	9
3	12 in, 34 lb	12	20.5	3⅝×⅜	8.6	10⅞
4	12 in, 39 lb	12	25	3⅝×⅜	8.6	10⅞
5	15 in, 39 lb	15	33	4⅛×⅜	9.2	13½
6	15 in, 45 lb	15	40	4⅛×⅜	9.2	13½

* Manufactured by the Carnegie Steel Company.

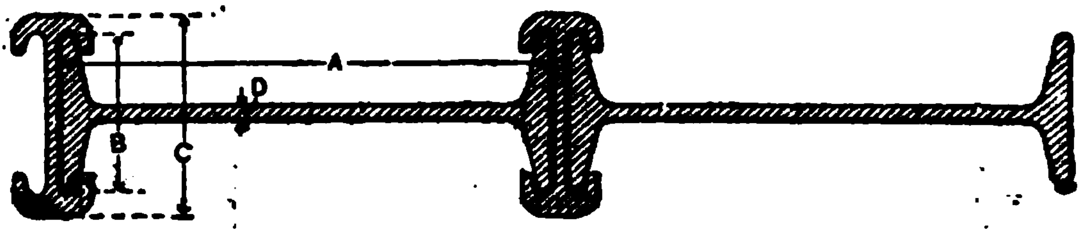


Fig. 41. Standard Sheet-piling

Standard Sheet-Piling *
Composition and Dimensions of Sections

No.	Size, in	Weight per square foot, lb	A	B	C	D
1	12×5	35.0	12	3.94	5	0.[illegible]
2	12×5	36.25	12	3.97	5	0.[illegible]
3	15×6	37.20	15	4.75	6	0.[illegible]
4	15×6	39.75	15	4.81	6	0.[illegible]
5	15×6	42.25	15	4.87	6	0.[illegible]

An interlocking bar is wedged to each beam at the mill and the two pieces are as a unit.

* Manufactured by the Jones & Laughlin Steel Company.

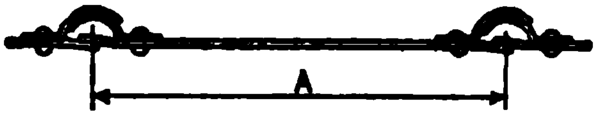


Fig. 42. Spring-lock Sheet-piling

Spring-Lock Sheet-Piling *
Composition and Dimensions of Sections

Distance, A	15¼ in	19¼ in	23¼
3/16-in plate, weight per square foot, in pounds.	17	14½	13
¼-in plate, weight per square foot, in pounds..	20	17	16

Plates may be obtained curved to any radius for circular work.

* Manufactured by the Mitchell-Tappen Company.

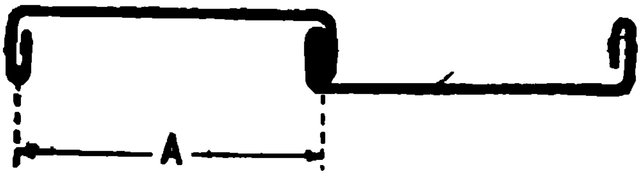


Fig. 43. Slip-joint Sheet-piling

Slip-Joint Steel Sheet-Piling *
Composition and Dimensions of Sections

Distance, A	6 in	9 in	12 in
No. 14-gauge, weight per square foot, in pounds.	5.4	5.8	5.7
No. 16-gauge, weight per square foot, in pounds.	4.3	4.0	3.9

* Manufactured by the Mitchell-Tappen Company.

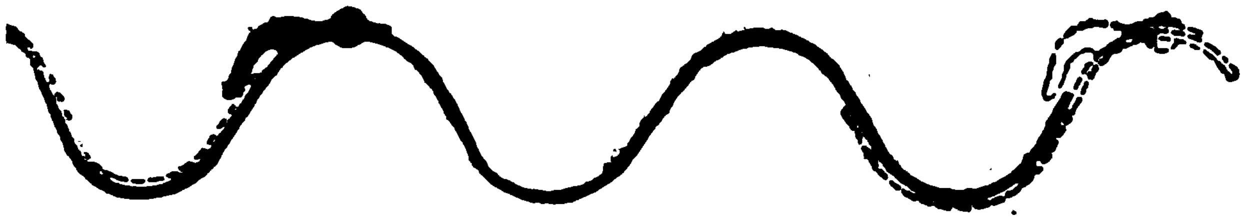


Fig. 44. Wemlinger Steel Sheet-piling

Wemlinger Steel Sheet-Piling *

Composition and Dimensions of Sections

......	1	2	3	4	5	6	7	8	9	10
pth of corrugation..............	2	2	2	2½	2½	2½	2½	4	4	4
ickness....................	1/16	7/64	1/8	7/64	1/8	5/32	3/16	3/16	1/4	5/16
dth, center to center of lap......	12	12	12	12	12	12	12	18	18	18
ight per square foot in pounds..	5	7.5	8.5	8	9.5	11.5	13.5	15	19	23.5

e dimensions given are in inches.

* Manufactured by the Wemlinger Steel Piling Company.

e Poling-Board or Chicago Method is a special method of excavation neral use in Chicago and in occasional use elsewhere for excavations which o a great depth in clay or in other suitable material. It has the advan- over the ordinary sheet-piling method that the lining of the excavation is driven. The method is not generally used for trenches or for square ex- tions as a circular excavation is more readily handled. The success of the hod depends entirely upon the character of the material to be encountered, he excavation is first made and the sides of the excavation afterwards orted. The method in detail for a circular excavation for a pier-foun- n may be described as follows:

) A circular excavation slightly in excess of the size required for the pier ried down to a depth of 5 ft, great care being taken to have the sides of excavation vertical and true to the circle.

) Vertical planks called LAGGING-PIECES, 5 ft in length and slightly beveled heir edges so that each piece may be considered as a stave with radial joints sponding to the size of the required circle, are set in place against the walls e excavation. These planks are held in place by two or more steel rings, ally made in quadrants, so that they may be conveniently handled and d together. The planks are wedged firmly against the walls of the excava- by means of wooden wedges driven between the planks and the iron rings.

) As soon as the first set of lagging is complete, the excavation is carried n for another section, 5 ft in depth, and another section of lagging is put in e and secured in the same manner.

pth and Character of Excavations in the Poling-Board Method. In the er described above the excavation may be carried down for an indefinite nce, a depth of 100 ft having frequently been attained. In many cases bottom of the excavation is BELLED OUT to a larger diameter than the ex- ion for the main shaft of the pier with the object of reducing the load of ndation-bed to a unit load less than the safe unit load on the main shaft of ier. This method is not adapted for running sand nor for clay that is not enough to stand with vertical sides during the necessary interval between g the excavation and placing the lagging. In some cases where a stratum uicksand has been encountered, the excavation has been carried past it

by the use of a cylindrical shell of steel, forced by jacks through it to an lying impervious layer of clay; but in general this method is dependent success upon a continuous body of impervious material.

The Open-Caisson Method or Well-Curb Method is used for pier carried to a considerable depth, and has advantages over the sheet-piling in certain materials. It is a development of the old method used in masonry wells and, in its modern form, consists of a structure which eve forms part of the pier itself and which is arranged with an open chambe base in which men may excavate the material under the structure and to settle as the excavation proceeds. It is evident that a central ope shaft must be left in the structure to permit of the passage of men and m

Details of the Open-Caisson Method. In detail, the method may be de as follows: First a CURB or CUTTING-EDGE of timber or steel, following t line of the pier, is constructed on the surface of the ground. The out of this curb is generally vertical and is protected with a steel plate wh tends below the main section of the curb, so as to form a cutting-edge o downward projection serving to penetrate the soil slightly in advance excavation. On this curb a wall of timber, concrete, or masonry is const inside of which the so-called WORKING-CHAMBER affords room for the w to be employed in excavating. Above the working-chamber the walls ma tinue to a height corresponding to the required height of the pier, leavi central space to be filled in after the required depth is reached; or a ro be built over the working-chamber and the entire cross-section of the pi with concrete or masonry excepting only a small central opening large en accommodate a HOISTING-TUB or BUCKET and to permit of the ingress and of the men employed in sinking the construction. In practice, the exca is started before the pier-structure is carried up to its final height, after the excavation and the building up of the pier progresses simultaneous constantly increasing weight of the structure aiding the sinking of th When the excavation has reached rock or a firm substratum, further e tion is stopped and the working-chamber and the central opening are full of concrete, leaving finally a complete pier-structure extending fr rock to the proper level to receive the steel grillage or other constructio ing on the pier.

Advantages of the Open-Caisson Method. This method of constructi the advantages that the workmen at all times are protected, that obstru such as boulders or logs, may be removed from under the cutting-edg that when rock is encountered, ample opportunity is afforded for the preparation of the rock-surface to receive the final concrete filling. If a mo amount of water is encountered, not accompanied by a flow of material, generally be taken care of by means of pumps.

Dredged Wells are similar to the open caissons described in the p paragraphs and are used where large quantities of water are encountered construction of the piers is similar to that of the piers used in the open- method; but the central shaft and working-chamber are designed to of the use of a CLAM-SHELL DREDGE or other form of dredge, and the w allowed to rise to its natural level in the working-chamber and shaft. method can be used to advantage when a considerable amount of water-b sand or other material is found overlying level rock or other firm found bed. When the dredging and the sinking of the pier-structure have been down to the hard underlying strata it is sometimes possible to pump o water. If this is not practicable the bottom may be prepared by divers reception of the concrete filling, and the concrete may be deposited through

...ing taken to use some special arrangement to protect the concrete from ...injured by loss of its cement-content, in the process of deposition.

...he **Well-Digger's Method** is also occasionally used in making PIT-EXCAVA... under walls or in cramped locations. By this method the sides of the ...ation are supported by planks placed horizontally. The method of plac... the planks is as follows: A shallow excavation, the depth of a plank, ...de by ordinary methods, and a SET, consisting of four planks fitting the ... sides of the excavation, is secured in place. Before proceeding with the ...al excavation of the pit a trench is dug directly alongside and underneath of the side planks of the FIRST SET. As soon as this trench is deep enough to ...modate the planks for the SECOND SET, the side of the trench under the ... already in place is cut to a vertical face, the plank placed in position and ...loose earth temporarily back-filled against it. As soon as the four planks ...ing the SECOND SET have been put in place by this method, the two side ...ks are wedged against the bank, the end-planks being used as struts. The ...planks are wedged into position and nailed or cleated to the side planks ...ing a PRESSURE-RESISTING FRAME supporting the side of the excavation. ...tinuation of this method enables the excavation to be carried on indef...ly, provided there is no flow of water or run of material causing an inflow ...terial into the excavation.

...he **Pneumatic-Caisson Method.** Where piers or foundation walls have ...e carried to a considerable depth through water-bearing materials, and espe...ly where large bodies of quicksand are encountered, the PNEUMATIC-CAISSON ...od must be resorted to. This method is based upon the PRINCIPLE OF A ...-BELL and may be briefly described as follows: The construction of the ...is similar to the piers previously described as used in the open-caisson and ...ed-well construction, except that the working-chamber and shaft are made ...ight and connected with a device called an AIR-LOCK, so that compressed air ... be introduced into the working-chamber. The object of the compressed ...is to prevent water entering into the working-chamber. This is accomplished ...cordance with the well-known PRINCIPLE OF THE DIVING-BELL by having ...compressed air constantly kept at a pressure which will counterbalance the ...-pressure at the level of the cutting-edge of the working-chamber. The ...ure of the air evidently must vary with the depth of the cutting-edge below ...-level. A column of water 1 in square in cross-section weighs .43⅓ lb per ...al ft, and it will therefore be counterbalanced by an air-pressure of .43⅓ ... sq in over the normal air-pressure. If the column of water is 30 ft in ...t, it will weigh thirty times .43⅓ lb, or will be counterbalanced by an air-...re of 13 lb per sq in above the atmospheric pressure.

...e **Maximum Air-Pressure in the Pneumatic Caisson** in which men can work ...ort periods is about 43 lb per sq in above atmospheric pressure, correspond...o a depth below water-level of about 100 ft. At this depth the work is ...d on in shifts of from two to three hours duration, and great care must ...ercised in coming out of the AIR-PRESSURE. The physiological effects of ...ssed air are often serious; pains in the joints, damage to the ear-drums ...g in deafness, and the so-called CAISSON-DISEASE render work at high ...e extremely hazardous.

...**Air-Lock Used in Connection with the Pneumatic Caisson** is a device for ...pose of retaining the air in the caisson and at the same time permitting ...age of men and material in and out. It consists essentially of a metallic ...ir CHAMBER or SHELL connected to the working-chamber either directly ...n air-tight lining or extension of the central shaft-opening. This air-...r has two doors, one at the bottom, opening downward into the shaft

and the other in the upper head of the air-lock chamber, also opening ward and affording a direct connection to the open air. In the operation air-lock one of these two doors must at all times be closed so as to preve free escape of air through the air-lock. If the bottom door is closed, it held firmly to its seat by the uplift of the compressed air in the shaft, wh at all times in direct communication with the working-chamber. If, under conditions, the upper door is open, the interior of the air-lock will be in communication with the open air and the air contained in the lock will evi be at atmospheric pressure. Workmen and materials may then enter t lock. In order to pass into the shaft and working-chamber, it is nece first, to close the upper door, and secondly, to shift the so-called EQUA VALVE and admit compressed air into the space between the two doors the air-pressure is brought up to the air-pressure in the working-chamb shaft. Pressure on the upper side of the lower door will then equal the p on the lower side and the lower door may be opened, the upper door being held against its seat by the compressed air in the air-lock. As soon as the door opens, the men and material may be passed into the shaft and wo chamber. In coming out the operations are reversed; men and material the air-lock through the open lower door, the lower door is closed and held against its seat, and the equalizing valve is shifted, affording a connecti tween the interior of the air-lock and the external air. The compress escapes through the equalizing valve, reducing the pressure in the air-l atmospheric pressure, and the upper door has atmospheric pressure o sides of it. It may then be opened, giving free connection with the outs

The Design of Pneumatic Caissons. The first consideration is, of cou have the final structure a permanent and sufficient pier to carry the loa imposed upon it. To this end the cross-section of the pier at all points fr to bottom should be capable of carrying safely the maximum load. cross-section of the pier is generally, in the finished pier, composed of sol crete, the cross-section will be determined by the allowable load on the co For piers the cross-section will generally be square or circular; for wa caisson will generally be not less than 6 ft in width, as it is difficult caissons having a width less than 6 ft. If the caisson is to be carried rock, the bearing on the rock need be no larger than the cross-section concrete pier; but if the excavation does not go to rock, it is frequently able to BELL OUT the base of the pier so as to reduce the loading on the tion bed to a unit load less than that allowable on concrete. The opera BELLING OUT is difficult in some materials; in a compact material it can erally accomplished without serious difficulty.

Piers Sunk by the Pneumatic-Caisson Method may be constructed of combinations of materials. The side walls and roof of the working-c were formerly frequently constructed of timber. In many cases they formed of steel; but in recent designs the working-chamber is generally of reinforced concrete, the only structural steel used being an angle or and angle composing the cutting-edge. The outside of the caisson is pr made vertical. The superimposed pier is generally of the same size as th ing-chamber, at least it is generally so in piers sunk for buildings.

A Typical Design for a Caisson Built of Reinforced Concrete is given in in which *AB* is the angle-iron and plate forming the so-called CUTTIN and *C* is the WORKING-CHAMBER formed by the side walls *DE* and *DE* the roof *EE*. The concrete side walls are reinforced with steel rods atta the cutting-edge, and extending upward into the body of the pier, and and body of the pier are reinforced to take care of stresses due to c

[illegible]sinking. In building up the working-chamber, the INTERIOR FORMS are [illegible] so as to support the concrete which makes the roof. These are sub-[illegible] removed. The exterior forms may constitute a permanent part of the [illegible], in which case they are called a COFFER-DAM, or they may be removed [illegible] the concrete has sufficiently set. At the center of the pier an open-[illegible] left to serve as the [illegible] opening connecting [illegible]ing-chamber with the [illegible]. The sides of this [illegible] of the upper part [illegible], are lined with an [illegible] STEEL SHELL. To [illegible] end of the steel [illegible] air-lock is connected. [illegible] of the pier does [illegible] 40 ft the construc-[illegible] may be completed [illegible] the excavation is com-[illegible]. Generally, however, [illegible]struction of the pier is [illegible] as soon as the work-[illegible]ber and from 5 to 10 [illegible] the superimposed pier [illegible] constructed; then [illegible] excavation is done, [illegible] the use of compressed [illegible] carry the cutting-edge [illegible] water-level. This is [illegible]ITCHING THE CAISSON [illegible] so that the caisson [illegible] some slight lateral [illegible] from the soil before [illegible]struction is carried up [illegible] to make it top-[illegible]. When the entire pier [illegible] first section is finished, [illegible] is resumed and the [illegible]structure is sunk as the [illegible] progresses, care be-[illegible] to remove any [illegible] from beneath the [illegible]edge. During the progress of sinking compressed air is conducted to [illegible]ing-chamber through the SUPPLY-PIPE *G*, the excavated material being [illegible] through the SHAFT *F*. The shaft *F* is fitted with a LADDER for the use [illegible]workmen.

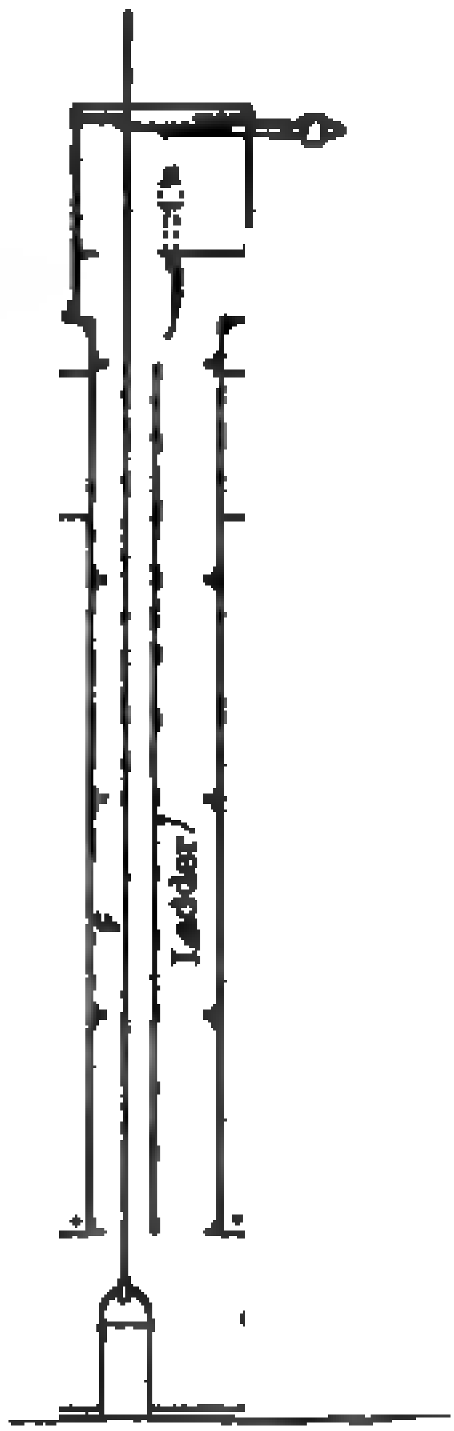


Fig. 45. Typical Form of Reinforced-concrete Caisson

[illegible] of Caisson-Sinking and Filling. In sinking the caisson and super-[illegible] pier, care must be taken to maintain it in a vertical position. This [illegible] be accomplished in large caissons by means of the excavation itself. [illegible] one side of the caisson is high the excavation on that side will be carried [illegible] in advance of the excavation on the low side, and the material under [illegible]-edge of the high side will be removed while a bank of material is [illegible] the cutting-edge of the low side. These methods, however, are of [illegible] when the caisson is narrow. In such cases that part of the caisson [illegible] above ground is held in position by GUIDES or other devices; but it

frequently happens that the caisson in its final condition is considerabl its correct location and considerably out of plumb. In general, there size of the caisson should be made larger than the minimum size nec order to allow for errors in its final location. When the caisson has rea required depth the foundation-bed is prepared for the reception of the filling and the working-chamber filled with it, care being taken that it co fills all voids and is in perfect contact with the roof. Finally, the air the steel lining of the shaft are removed and the shaft-opening filled crete to the proper level to receive the GRILLAGE or other construction the base of the column which is to rest on the caisson.

The Height of Caisson-Piers. The height of a pier cannot be fixed until it is known to what depth the caisson must sink in order the foundation-bed. If the rock is found at a greater depth than ant additional height is added to the top of the pier after the caisson is in position; but if, on the other hand, the rock is found unexpectedly l top of the pier will have to be cut off. If the finished elevation of the be below the level of the general excavation, it is usual to extend the surface of the pier to the required height by means of a temporary structure called a COFFER-DAM, the height of which corresponds to the the finished surface below the level of the general excavation. Insid COFFER-DAM some STEEL GRILLAGES may conveniently be set.

The Freezing Process for Excavations. This method has s been employed in making excavations. In this country its use has bee to one or two mining-shafts, but in Germany it has been resorted to in excavations for building-foundations. The method consists in driv pipes into the ground. These pipes are closed at the bottom and at are connected to smaller pipes through which brine, at an extremely l perature, is made to circulate. The refrigerating effect results in free water contained in the soil, converting quicksand to a frozen mass re soft sandstone. When the freezing has progressed sufficiently to form wall or coffer-dam around the excavation, the material inside the fro may be excavated. This method has the advantage, theoretically, applicable to excavations of any depth. There are many precautions n and for the present, at any rate, it should only be considered as a possil

31. Protection of Adjoining Structures

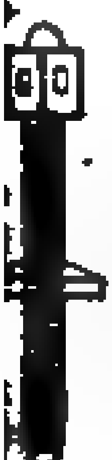
General Considerations. The COMMON LAW provides that any pers ing an excavation is responsible for resulting damage to adjoining STATUTE LAWS as embodied in the building codes of different cities ma or limit this responsibility, but in general, excavations should be mad a manner as to cause the least possible damage to surrounding property. there are no adjoining structures it is generally sufficient to slope the sid excavation so as to prevent the sliding of material into the excavatio least, to sheet-pile and brace the sides of the excavation; but where the tion is to be made alongside of an existing structure, and carried b footings of such structure, it is necessary to take special measures for it tion. Such work is described as SHORING, UNDERPINNING and PROTEC JOINING STRUCTURES, and may involve the carrying of the weight of p of the buildings on temporary supports, the removal of the old foot the construction of new footings at lower elevations.

Shoring. When the excavation for the new building does not below the adjoining footings and when the material is fairly solid, it m to transfer a portion of the load of the wall to temporary footings. Thi

plished by means of heavy inclined posts called SHORES, arranged to act
...ing COLUMNS or STRUTS. Each SHORE consists of a post, the lower
... which rests on a PLATFORM, generally consisting of planks and timbers
... so as to form a temporary spread footing. This platform should be
... at a depth which will insure that subsequent operations will not under-
...it. The upper end of the post fits into a hole or niche cut into the wall
... supported. The post itself may be a timber with a square cross-section,
... 12 by 12 in. and of the required length. Provision is made, between the
... and the lower end of the post, for WEDGES or JACKS, so that when
...ed their lifting effect transfers part of the weight of the wall from its
... to the temporary foundation or platform. During this operation all
... of the temporary structure are in compression and brought into bearing,
... material under the platform is compressed and solidified as much as
...le.

...s of Shores. If the SHORE is to act preferably for LIFTING only, it is
... nearly vertical as possible and is known as a LIFTING SHORE. If it is
... preferably to combine a horizontal PUSHING action with the lifting
..., it is placed at a considerable angle from the vertical and is then known
...SHING SHORE or STEADYING SHORE. In arranging such shores care
... be taken to have the niche cut close to a floor-level of the building to be
..., as otherwise the horizontal component of the thrust of the shores might
... the wall.

...s and Sizes of Shores. Where a wall is light, a number of smaller ... should be used in preference to a few large ones. Where a wall is high, ...r more shores of varying ... may be used, and these ...conveniently be placed in the ... vertical plane and rest on the same platform.



... Standard ... of Steel ...-jack

Wedges and Screw-Jacks. In transferring the load of a wall from its own footing to the temporary platform, use is made of wooden or steel WEDGES, SCREW-JACKS, or HYDRAULIC

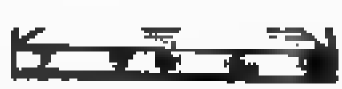


Fig. 47. Standard Type of Steel Screw-jack

...s, wedges and jacks may be used in combination. Wooden wedges ... be made of hard wood and are generally arranged in pairs, both ... being driven at the same time. The lifting effect of such wooden ... is powerful, but where a considerable settlement of the temporary ...m is anticipated, it is more convenient to use screw-jacks, as they ... up a considerable settlement.

...ls and Types of Screw-Jacks. The SCREW-JACKS usually manu... for this purpose are made of cast iron and have rough threads, with ... a pitch to have much lifting effect. Screw-jacks of a better kind are ...eel and have a machine-thread of small pitch. Such jacks can be ...pable of lifting weights up to 100 tons. Figs. 46 and 47 represent

standard forms of screw-jacks. When a single screw-jack is used in
tion with a post or shore, a hole to receive the threaded portion of
is bored in the end of the timber used for the shore, the end being sq
receive the nut. Such an arrangement is called a PUMP and is illust
Fig. 48. When a lifting effect greater than that exerted by a single ja
quired, the jacks are arranged in pairs in connection with a short timber c

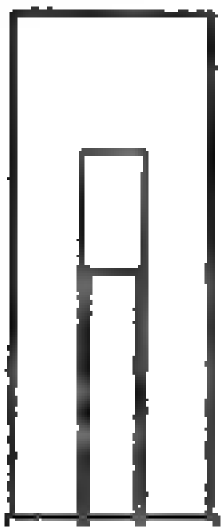


Fig. 48. Pump, or Screw-jack let into End of Shore

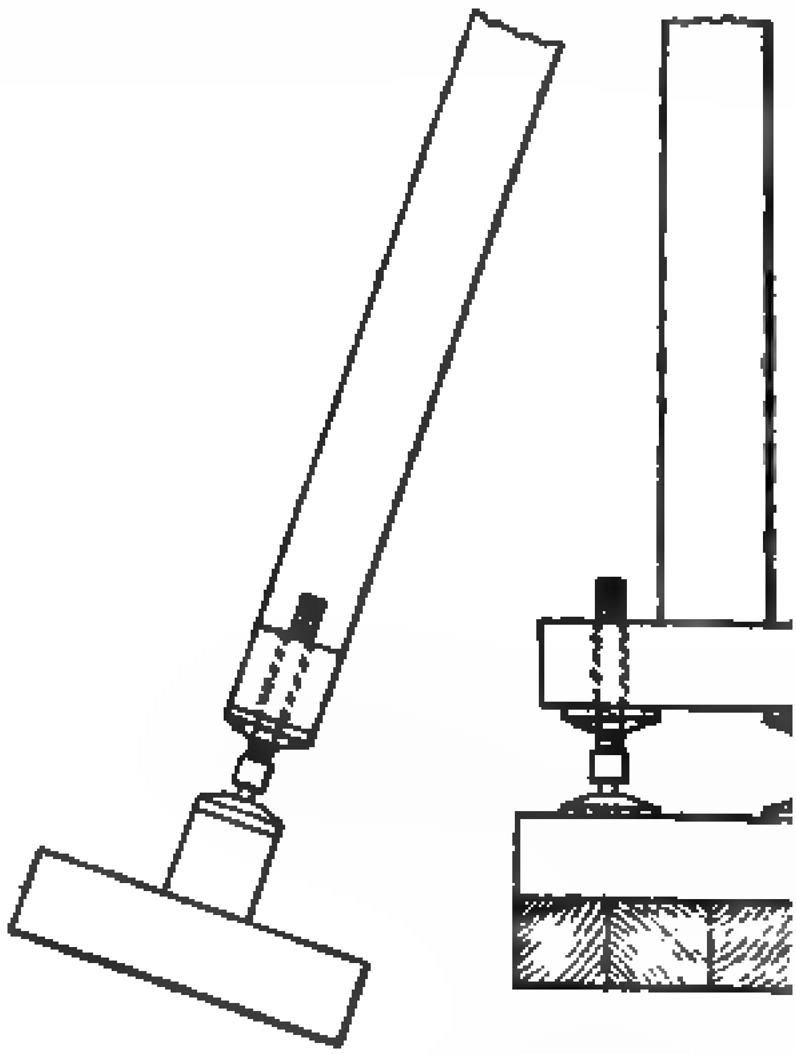


Fig. 49. Shore, Screw-jacks and Timber head

HEAD. Such an arrangement is illustrated in Fig. 49. It has the ac
that after operating the jacks, blocking and wedges can be placed betv
platform-timbers and the cross-head so that the post resting on the cr
has a direct and solid bearing on the platform. By this method the loa
wall can be thrown on the platform by the jacks and after the blocking an
ing is in position the jacks can be removed.

Hydraulic Jacks. Where excessively heavy loads are to be lifted, HY
JACKS may be used in place of screw-jacks but an objection to them is th
are liable to SLACK BACK under the load. While the load, therefore
not be permanently supported on hydraulic jacks, they may be used to
load temporarily while the blocking and wedging are being placed betw
cross-head and the temporary footing. In this way an indefinite nu
shores may be set and taken care of with a single pair of hydraulic jacks

Example of Shoring. Fig. 50 shows the method used in SHORING t
mental front wall of a heavy building, advantage having been taken of
merous deep margin-drafts shown in the section. In order to avoid the
of cutting niches for the tops of the shores, nine hardwood blocks, a,
were fitted to the margin-draft grooves in the masonry. Nine similar
b, b, etc., were gained into and bolted to the vertical timber, V V, spa

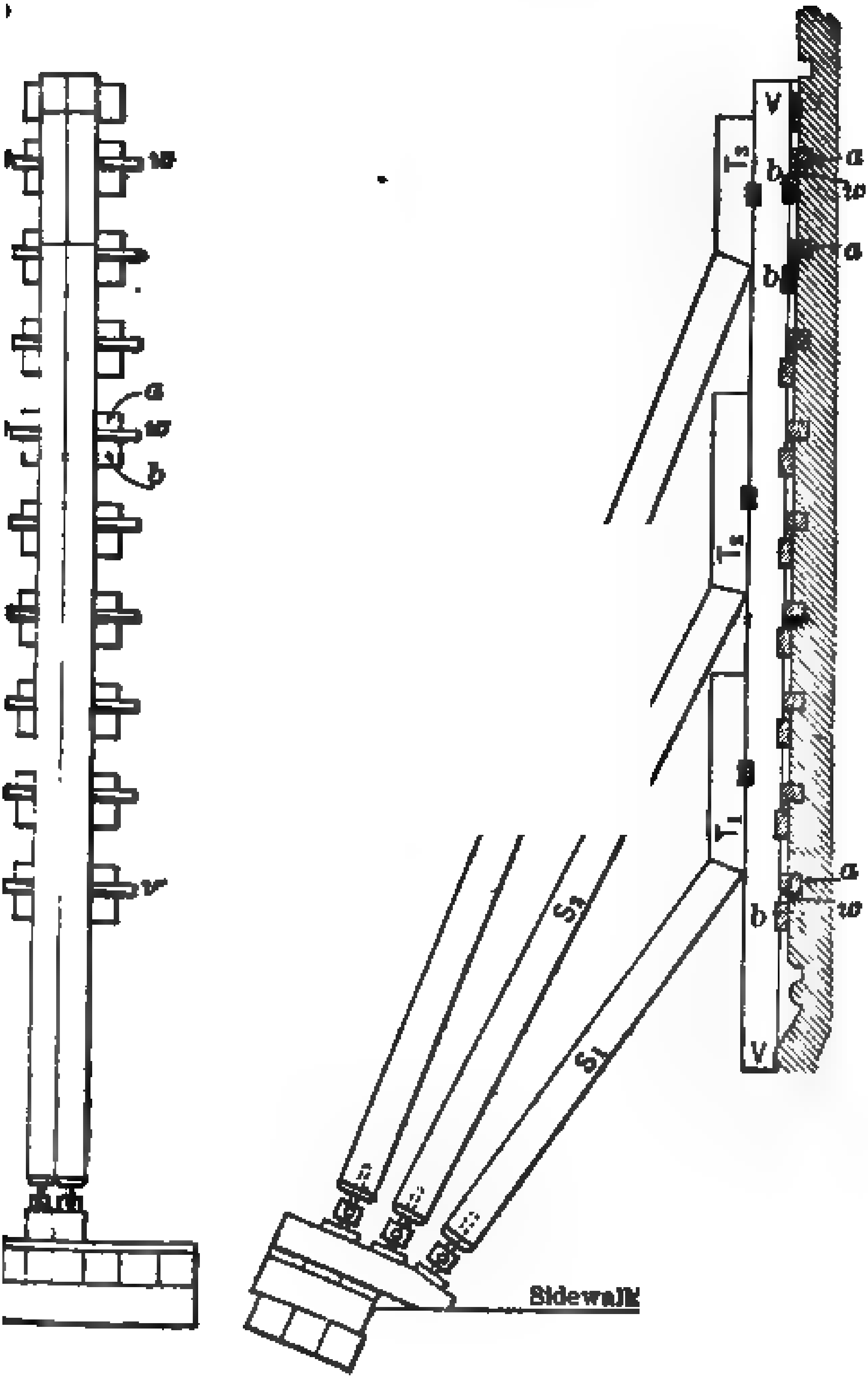


Fig. 50. Shoring an Ornamented Wall

[illegible] the *a* blocks and the *b* blocks for adjusting wedges *w*, *w*, etc. Three [illegible], T_1, T_2 and T_3 were keyed and bolted to *VV* and transmitted to it [illegible] of the three shores, S_1, S_2 and S_3. Each shore had a 60-ton screw-jack [illegible]. Each shore is shown fitted with a pump or detached extension-[illegible] for the screw-jack.

Needling. NEEDLES or GIRDERS are employed when part or all of th of the wall has to be carried, as when the old footing is to be removed wall UNDERPINNED or carried down to a new footing at a greater depth.

Example of Needling and Underpinning. Fig. 51 represents a typica UNDERPINNING, the several operations being as follows:

(1) **The General Excavation** is carried down to within a few inche bottom of the footing *BB* under the wall *W*.

(2) **The Pit** *DDDD*, properly braced and protected by sheet piling, to approximately the level of the proposed excavation, this PIT bein at a safe distance from the existing wall. In good material it may be

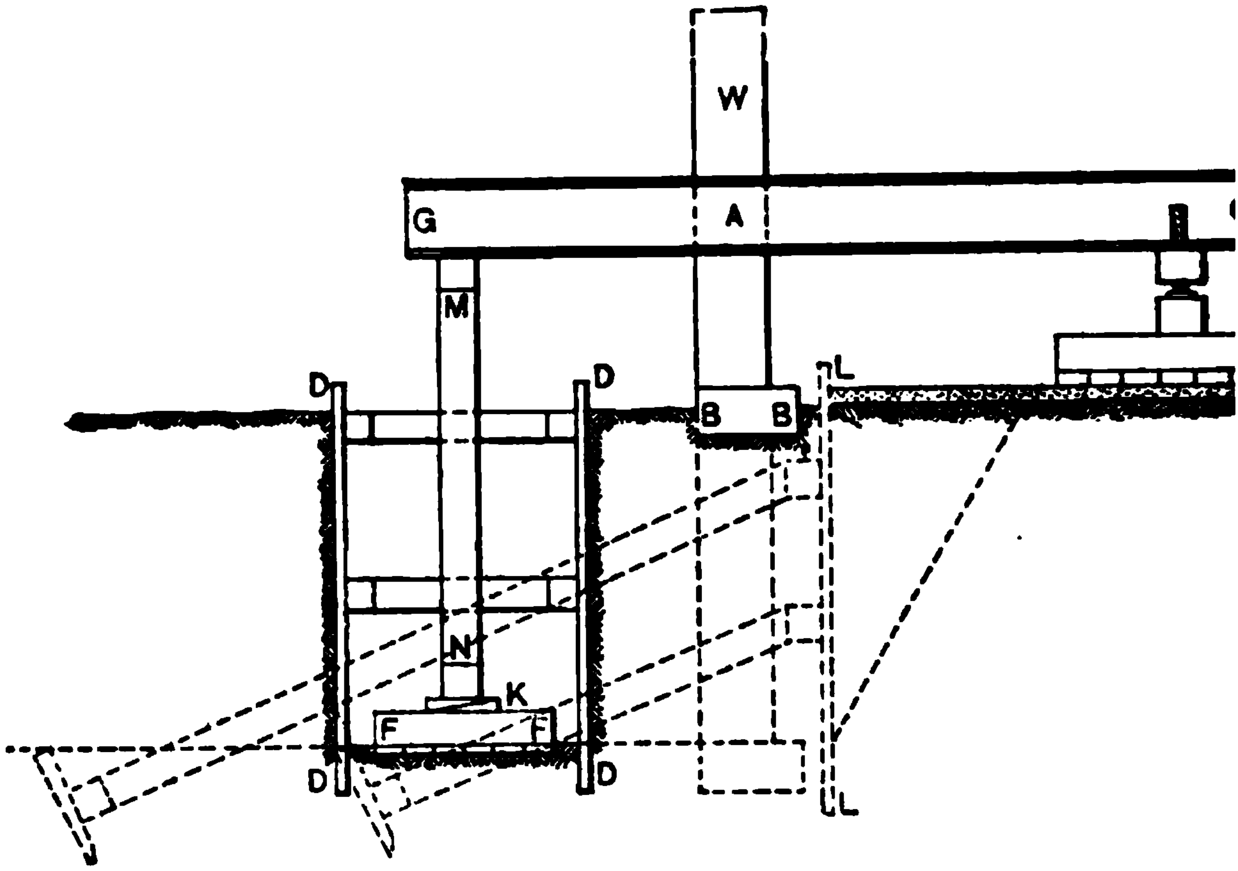


Fig. 51. Wall-needling and Underpinning

have this pit approach to within a few feet of the footing course of the w in material which is liable to run it should not approach the wall clos its depth. No hard and fast rule can be given, and in every case gr should be taken to prevent any movement of the material from under joining footing.

(3) **The Platforms.** On the bottom of this pit-excavation, a PLATF is placed, generally composed of heavy timbers resting on a base o planks, and acting as a support for the outer end of the needle. Du construction of this pit a similar pit may be dug on the inside of the wal vide for the support of the inside end of the needle; but as this invol destruction of the cellar-floor the method of procedure inside the bui generally different from this. If the material is solid it is sometimes s to place the platform for the support of the inside end of the needle on the cellar-floor and at such a distance from the wall that the n excavation for the new footing will not disturb it; or the platform placed on the cellar-floor and a line of sheeting *LL*, properly braced, so that the excavation can be made for the new footing. This is generally s to prevent any serious settlement of the temporary platform for the in of the needle.

[The] Insertion of the Needles. Having provided a support for each end [of the ne]edles it only remains to cut a hole through the wall, as at *A*, insert the [needle] *GG*, put the post and blocking *MN* under the outside end of the needle, [and th]e blocking and jacks under the inside end. The post *MN* may be fitted [with we]dges as shown at *K*, or with one or more screw-jacks. The needle *GG* [may con]sist of one or more heavy timbers or one or more steel I beams. In any [case th]e load to come on this needle should be figured and its strength made [sufficien]t to safely support such load. As soon as the weight of the wall *W* is [transfer]red to the needles and to the temporary platforms prepared to receive [it], that part of the wall which is below the needles and all of the foot[ings ma]y be removed and all of the excavation for the new footing made.

the footing is removed. The hanging parts of the wall may be removed pended by rods and chains to the needles. If they are not so susp crack will form along the line *AAAAA*.

Transferring the Load to the New Underpinning. As soon as footing has been put in place and the new wall carried up ready to rec old wall, provision must be made for REVERSING THE OPERATION, tha transferring the load onto the new underpinning wall and footing. generally done by means of a number of GRANITE BLOCKS set in pairs the needles and fitted with STEEL WEDGES. After setting these blocks, t between the base of the old wall and the top of the upper wedging block in with brickwo mortar in the la being compacted b of PIECES OF SLAT in so as to we mortar between th This brickwork sh laid up in Portland mortar so as to re time of setting. as it is sufficiently wedges are driven as to throw at leas tion of the weigh wall on the new As a result of thi quently happens t footing settles, t being restored needles. This nec continued driving wedges until it has its final settlemen will be evidenced b ing of the wall suff partially relieve th in the needles and fact that the we main tight.

Fig. 53. The Figure-four Method of Needling

Removal o Needles, etc. as the entire weigh wall has been tr to the footing and the footing has demonstrated that it is capable porting the weight of the wall without further settlement, all of the te work, including the needles, can be removed, the needle-holes bricked the repairs made to the cellar of the adjoining building.

The Figure-Four Method of Needling. In certain cases it is im to employ a NEEDLE-BEAM projecting on both sides of the wall, as for when the occupancy of the adjoining building is such as to make it im to have a needle-beam projecting into the cellar space. In such cases called FIGURE-FOUR NEEDLE has been employed (Fig. 53). In this needle *AB* acts as a CANTILEVER. Part of the load of the wall is carrie

Fig. 54. The Spring-needle Method of Underpinning

tending to lift the needle-beam acts on both walls, but on account of its [illegible]located nearer to the wall to be lifted, a large proportion of its effect is [illegible] thereon.

[illegible] or Cylinders for Underpinning are frequently used for the support [illegible] wall and have many advantages, as they not only afford a support for the [illegible] through the operations affecting the stability of the wall, but also form [illegible]ment support. The operation in brief is as follows: A hole or niche is [illegible] the wall and footing to be supported, of sufficient size to permit the intro[illegible] of a section of STEEL PIPE, in such manner that the center of the pipe [illegible] below the center of the wall to be supported, the height being sufficient [illegible]commodate a section of pipe and also the means employed to drive it. [illegible] pipe may be driven (1) by HYDRAULIC JACKS or by SCREW-JACKS, placed [illegible] the top of the pipe and the wall itself, as by the patented BREUCHAUD [illegible]; (2) it may be driven by means of a POWER-HAMMER driven either by [illegible] compressed air; or (3) in some cases, where the material is fine sand or [illegible] the pipe may be JETTED or the JET-METHOD may be used in combination [illegible] either jacks or power-hammers. In any case the first section of pipe is [illegible] into the ground and additional sections are added until the lower end of [illegible] encounters rock or some material possessing sufficient stability to insure [illegible] required support. The material entering the pipe is removed by a WATER-[illegible] by other means and the space filled with concrete. As soon as the con[illegible] set sufficiently the pipe is capped with a special casting on which short [illegible] beams are arranged to distribute the support of the pipe over a consider[illegible] of the base of the wall to be supported These I beams correspond

to the wedging blocks used in the ordinary methods before described.] vision is frequently made for STEEL WEDGES between the cap and the base of steel beam, but it is generally found sufficient to thoroughly grout in the between the base of the wall and the steel beams after the niche itself has bricked up.

Cylinders for Underpinning Very Heavy Walls. The description i preceding paragraph is intended to cover the use of pipes varying in size 6 to 20-in in diameter, according to the load to be carried. In the case of e sively heavy walls, CAST-IRON CYLINDERS are used in place of steel pipes. cylinders are arranged in sections, each section making a water-tight joint the preceding section, and are generally used where water is encountered where it is necessary to carry down the underpinning to rock at great de Under such conditions these cylinders are sunk by the PNEUMATIC-CA METHOD. Such cylinders have been sunk to a depth of 70 ft below water and have been designed to carry as much as 400 tons.

CHAPTER III

...NRY WALLS. FOOTINGS FOR LIGHT BUILDINGS.* CEMENTS AND CONCRETES

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1. Footings for Light Buildings

...ing Courses in General.† Every foundation or bearing wall overlying ...ing except solid rock should rest on a footing or base projecting beyond ...l on each side. On wet or very compressible soils these footings may be ...f steel beams or of reinforced concrete, as described in Chapter II, but ...nably firm soils and for buildings of moderate size and weight the foot-...re generally of concrete, stone, or brick. Footings answer two important ...es:

By distributing the weight of a structure over a larger area of bearing ...l, the pressure per square foot on the foundation-bed is diminished and ...ndency to vertical settlement correspondingly lessened.

By increasing the area of the base of a wall, footings add to its stability ...rm a protection against the danger of the work being thrown out of ...by any forces that may act on it. Nearly every building law requires ...very foundation wall or pier and every cellar or basement wall or pier ...ave a footing at least 12 in wider, that is, 6 in on each side, than the thick-...f the wall or pier, and this may be considered as the minimum projection, ...t in rare instances where there may be a special reason for making it less. ...n soils and for comparatively light buildings a projection of 6 in on each ...f a wall will generally reduce the unit pressure, that is, the pressure per ...foot, to the safe resistance of the soil, but it is always wise to proportion ...otings to a uniform unit pressure, as explained in Chapter II, Subdivision ...o have any useful effect, footings must be well bedded and have sufficient ...erse strength to resist the upward reactions on the projections.

...e Footings for Walls with Ordinary Loads. Stone cellar walls and ...ent walls generally have stone footings, although if the walls are heavily ...l a bottom footing of coarse concrete is advisable under the stone footing. ...ticable, stone footings should consist of stones having a width equal to ...f the footing. If impracticable to obtain stones of this size, then two ...should be used, meeting under the middle line of the wall. In any event ...ting course should extend inside of the course above, a distance equal to ...t one and one-half times the projection, otherwise the stones will not ...ly transmit the loads and reactions and the footing courses will tend to ...t the joints, as in Fig. 1.

...r a complete discussion of foundations in general and the mechanical principles ...d in their strength and stability, for walls, piers, etc., below the basement or ...ler, see Chapter II.

...r a complete discussion of footing courses for heavy buildings and of the theories ...resses developed in offset, projecting, or cantilever footings, see Chapter II. ...ly Subdivisions 17 to 23.

Stone footings should be of hard, strong and durable stones, always their natural bed and solidly bedded in mortar. As a general rule, f buildings, and where the loads per unit of foundation-bed are much le the allowable pressure, the thickness course is made about equal to its pr beyond the course above. The mo mon defect in large-stone footings the stones are not properly bedded, more difficult to bed a large stone small one. The stones should be la thick bed of mortar and worked sidew a bar until firmly settled in place.

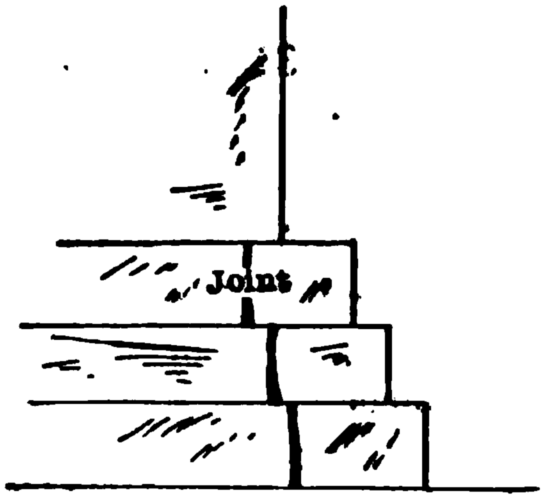

Fig. 1. Stone Footing. Openings at Joints

Offsets in Masonry Footings. projections of the footing courses the wall or beyond the courses abov be carefully considered, whatever t terial of the footings. If the proje the footing or offset of the courses great for the strength of the stone, brick, or concrete, the footing wil as shown in Fig. 2. The proper offset for each course depends u vertical load, the transverse strength of the material, the resisting power of the foundation-bed and the thickness of the course.†

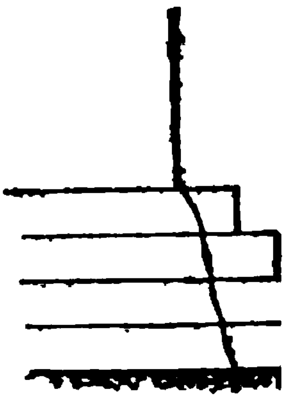

Fig. 2. Crack in from Excessive C

Tables for Offsets for Masonry Footing Courses.‡ As stated in Chapter II, in the discussion of the design of stepped footings, there are rule-of-thumb methods giving so-called safe projections for given depths of footings or giving the ratios between the projections and the depths of the courses. Tables of offsets for footing courses of different materials have been computed from the flexure-formula applied to the projecting footing courses considered as CANTILEVER BEAMS uniformly loaded by the upward pressures on the un Although these tables, so computed, are incorporated in some buildin they cannot be safely used without numerous restrictions, exceptions an fications, and hence they are, in general, unreliable and of use only as mations. As these tables are still inserted in engineers' and architect books, the table of offsets for masonry footing courses, in a revised for tained in this chapter with the recommendation that for footings of offsets it be used with caution and that for such footings the methods e in Chapter II be used when greater accuracy of results is required.

Notes Regarding Use of Table I. The values in Table I are co from the formula $l = \frac{1}{6} d \sqrt{S_f/wf}$ which is derived from the FLEXURE-F for a uniformly loaded cantilever beam, and slightly changed to m numerical coefficient of the second member of the equation the value In this equation, l = the maximum allowed offset of the footing course in d = the thickness of the footing course in inches, S_f = the modulus of

* See Offset Footings, Chapter II, especially Subdivisions 17 and 22.

† See Chapter II, Subdivision 22, for a complete discussion of the principles in the design of projecting footings, ratio of projection to depth of footing, etc., geneous slabs, separate-layer footings, etc.

‡ See Chapter II, Subdivision 22, page 180.

§ See, also, formula in Chapter II, Subdivision 22.

ble I. Approximate Values of Offsets for Masonry Footing Courses in Terms of the Thickness of the Course

The values are computed with a factor of safety of 10.

Material of the footings	S_f in pounds per square inch	w in tons per square foot 0.5	1.0	2.0
rth River Bluestone (ordinary run)	3 000	4.1	2.9	2.0
anite (average)	1 850	3.2	2.2	1.6
mestone (average)	1 375	2.8	2.0	1.4
ndstone (average)	1 375	2.8	2.0	1.4
ickwork (good bricks in natural-cement mortar, 1:2, 60 days old)	125	0.8	0.6	0.4
ickwork (hard-burned bricks in Portland-cement mortar, 1:3, 60 days old)	400	1.6	1.1	0.7
ncrete (Portland cement, 1 : 2 : 4, 1 month old)	300	1.3	0.9	0.6
ncrete (Portland cement, 1 : 2 : 4, 6 months old)	400	1.6	1.1	0.7

the materials in pounds per square inch, w = the determined or assumed sure on the bottom surface of the footing course considered, in tons of 2 000 per sq ft, and f = the factor of safety used. The table gives the values of l/d three unit pressures w. For example, if w is taken at 2 tons per sq ft, then limestone or sandstone footings l/d = 1.4, and if d, the thickness of the ing course, is 12 in, the offset or projection should be 16 or 17 in. The es given in the table for S_f, the modulus of rupture for the materials, differ slightly from those given in Subdivision 22 of Chapter II, in Table I of pter XV and in Table III of Chapter XVI. If results are required based different fiber-stresses, upon a different factor of safety, or upon different sures per square foot, the formula may be used instead of the table. It ld always be borne in mind that as each footing course transmits the entire ht of the wall and its load, the pressure will be greater per square foot on upper courses, and that the offsets should be made proportionately less; that the values in Table I, when applied to stone-masonry footings, are y valid for the lower offset only, and then only when the footing is built nes the thickness of which is equal to the thickness of the course, which e a projection of less than half their length, and which are well bedded in nt mortar.

ncrete Footings.* For buildings of great weight, except the very heaviest, especially for those built on a clay soil, concrete generally makes the best ing, and it is even preferable to and generally cheaper than large slabs of e. When the concrete is properly made and used, it attains a strength l to that of most stones, and under walls, being devoid of joints, it is like tinuous BEAM, having sufficient strength to span any soft spots that pen to be in the foundation-bed. When deposited in thin layers and well ed, concrete becomes firmly bedded on the bottom of the trenches, so that e is no possible chance for settlement except that due to the compression of oil.

* For an example of concrete-footing design, see Chapter II, Subdivision 22. For ed-concrete-footing design, see Chapter II, Subdivision 24. See, also, Chapter paragraphs relating to footings, pages 978 to 982.

Preparing the Trenches. For footings, concrete made with Po cement is preferable, and it should have a thickness of at least 8 in, even light buildings; and for buildings of more than two stories, a thickness least 12 in. On firm soils, such as hard clay, the trenches should be accu dug and trimmed to the exact width of the footings, so that the concret fill them. When the foundation-bed is of loose gravel or sand it is gen necessary to set up planks to confine the concrete and form the sides footings. These planks may be held in place by stakes; they should b in place until the concrete has become hard, which generally requires fro to four days, after which they may be pulled up and dirt filled in again concrete. The proportions and manner of mixing concrete are described latter part of this chapter.

Depositing the Concrete. Concrete should be used as soon as mixe should always be deposited in layers, which as a rule should not exceed 6 thickness, especially for the first layer. On small jobs where the work i by hand the concrete is usually carried to the trenches in wheel-barrow dumped into the trenches. The height from which the concrete is du however, should not exceed 4 ft above the bottom of the trench, because w falls from a greater height the heavy particles are apt to separate fro lighter ones. As soon as the concrete has been deposited in the trenc should be leveled off and then tamped with a wooden rammer weighing 20 lb, until the water in the concrete is brought to the surface. Concrete not be permitted to dry too quickly, and if twenty-four hours elapse be the deposits of the successive layers, the top of each layer should be spr before the next is put in place. For buildings over five stories high, it is idea to place a stone footing course above the concrete footing, if suitable for the purpose can be obtained.

Brick Footings. Where the foundation walls are of bricks, the footi usually brick or concrete. For interior walls on dry ground, and in

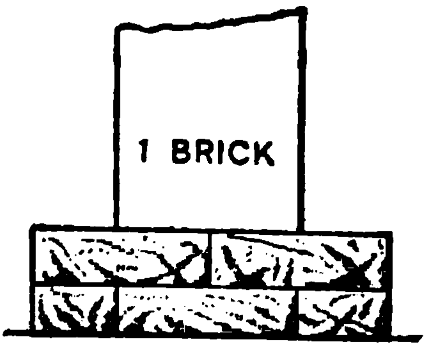

Fig. 3. Brick Footing. Wall One Brick Thick

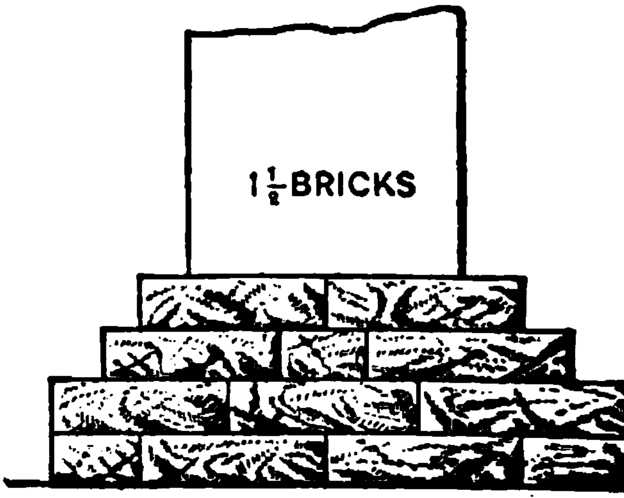

Fig. 4. Brick Footing. Wall One One-half Bricks Thick

localities for outside walls, brick footings are fully as good as stone fo provided good, hard bricks are used and the footings are properly built. footings should always start with a DOUBLE COURSE on the foundation-b then be laid in single course for ordinary footings, the outside of the wor laid all HEADERS, as in the accompanying illustrations, and no course pro more than one-fourth the length of a brick beyond the one above it, ex the case of an 8-in or 9-in wall. For brick footings under high or heavily walls, each projecting course should be made double, the HEADER-COURSE and the STRETCHER-COURSE below. Figs. 3, 4, 5 and 6 show footings fo

ing from one brick to three bricks in thickness. The bricks used for footshould be the hardest and soundest that can be obtained, should be laid in t mortar and should be either grouted or thoroughly slushed up, so that joint shall be entirely filled with mortar. The writer favors GROUTING brick footings, that is, the g of a thin mortar to fill the e joints, as he has always d that it gives very satisfacresults. The bottom course the footing should always be in a bed of mortar spread the bottom of the trench the latter has been carey leveled. All bricks laid in m or dry weather should be oughly wet before laying, for, laid dry, they rob the mortar a large percentage of the mois-

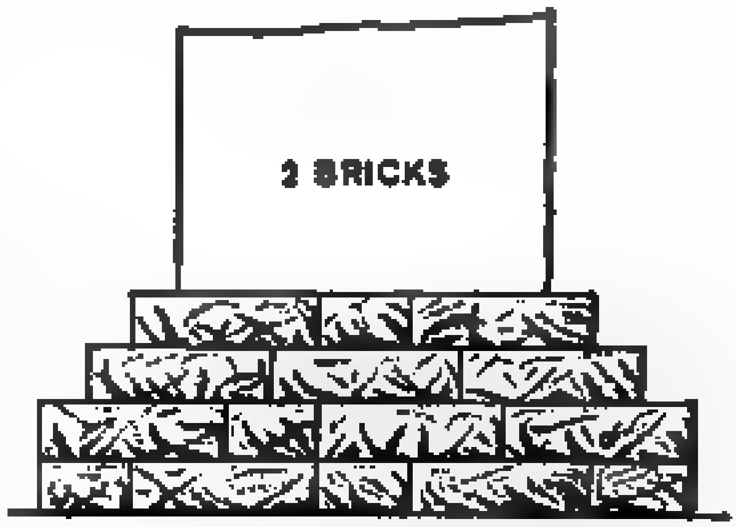

Fig. 5. Brick Footing. Wall Two Bricks Thick

it contains, greatly weakening the adhesion and strength of the mortar. ful attention should be given to the laying of the footing courses of ings, as upon them the stability of the work largely depends. If the m courses are not solidly bedded, if any rents or voids are left in the of the masonry, or if the materials themselves are unsound or badly

Fig. 6. Brick Footing. Wall Three Bricks Thick

together, defects in the superstructure are almost sure to show themp sooner or later, and almost always at a period when remedial efforts are ult and expensive.

erted Arches.* In a few buildings in which the external walls are ded into piers with wide openings between them, and in which the supportwer of the soil is not more than 2 or 3 tons per sq ft, it was thought desirconnect the bases of the piers by means of INVERTED ARCHES, for the purf distributing the weight of the piers over the whole length of the footings. les of inverted-arch footings are shown in Figs. 7 † and 8,† which represent ively the construction employed in the Drexel Building in Philadelphia

r an example worked out in full, showing the method of proportioning inverted see Chapter III, Building Construction and Superintendence, Part I, Masons' y F. E. Kidder.

m the Engineering Record, May, 1899, and Nov., 1890.

and the World Building in New York City. Unless the piers are about eq loaded, however, it is generally impossible to distribute the weight evenly if the arches extend to an angle of the building, the end-arch must be pro with ties of sufficient strength to resist the THRUST of the arch, as otherw may push out the corner-pier. It is usually better to build the piers with rate footings, projecting equally on all sides of the pier, and each proport

Fig. 7. Inverted-arch Footing. Drexel Building, Philadelphia

Fig. 8. Inverted-arch Footing. Building, New York

to the load supported. The intermediate wall may be supported by steel or by arches. About the only advantage over ordinary masonry fo possessed by inverted arches is in the resulting shallower foundations.

The following, relating to inverted arches, is taken from the New York ing law: "If, in place of a continuous foundation wall, isolated piers are built to support the superstructure, where the nature of the ground ar character of the building make it necessary, in the opinion of the Commis of Buildings having jurisdiction, inverted arches resting on a proper b concrete, both designed to transmit with safety the superimposed loads, be turned between the piers. The thrust of the outer piers shall be tak by suitable wrought-iron or steel rods and plates." (Law of 1906.)

2. Cellar Walls and Basement Walls

Definitions. These terms are generally applied to walls which are the surface of the ground or below the water-table or first-floor l which support the superstructure and which go down to the FOUNDATION v properly so called. (See Chapter II, Divisions 1 and 29.) Walls whose office is to withhold a bank of earth, such as the walls around areas, are RETAINING-WALLS. (For retaining-walls, see Chapter IV.)

Materials for Cellar and Basement Walls. These walls may be b brick, stone or concrete. BRICK is suitable only in very dry soils or for a wall with a cellar or basement on each side of it. PORTLAND-CEMENT CON is an excellent material for foundation walls, and is being more exte used in their construction every year. The concrete may be filled in b wooden forms, which hold it in place until it has set, or concrete blocks s so as to form a solid wall may be used. If POURED CONCRETE is used the

... be removed as soon as the concrete has set and the walls should be ... once or twice a day, if the weather is dry, so that the concrete will not ... quickly Good hard LEDGE-STONE, especially if it comes from the quarry ... flat beds, makes not only a strong wall but, if well built, one that will stand ... effects of moisture and the pressure of the earth much better than a brick ... Between a good stone wall and a wall of Portland-cement concrete, ... is probably not much choice, except perhaps in the matter of expense, the ... relative cost of stonework and concrete varying in different localities. A wall ... of soft stones, or stones that are very irregular in shape, with no flat surfaces, ... greatly inferior to a concrete wall, or even to a wall of good hard bricks, and ... should be used only for dwellings or light buildings. Stone walls should never ... less than 18 in thick, and should be well bonded, with full and three-quarter ... headers, and all spaces between the stones should be filled solid with mortar ... broken stones or spalls. The MORTAR for stonework should be made of ... cement and sharp and rather coarse sand. The outside walls of cellars ... basements should be PLASTERED smooth on the outside with 1 : 2, or 1 : 1½ ... cement mortar, from ½ to ¾ in thick. In heavy-clay soils it is a good idea to ... the walls on the outside, making them from 6 in to 1 ft thicker at the ... than at the top.

Thickness of Cellar and Basement Walls. This is usually governed by ... of the walls above, and also by the depth of the wall. Nearly all building ... regulations require that the thickness of the cellar and basement wall, to the ... depth of 12 ft below the grade-line, shall be 4 in greater than the thickness of the ... above for bricks, and 8 in greater for stone, and that for every additional ... or part thereof in depth, the thickness shall be increased 4 in. In all large ... the thickness of the walls of buildings is controlled by law. For buildings ... which the thickness of the walls is not so governed, the following table will ... as a guide:

Table II. Thickness of Cellar and Basement Walls

Height of building	Dwellings, hotels, etc.		Warehouses	
	Brick, in	Stone, in	Brick, in	Stone, in
... stories	12 or 16	20	16	20
... stories	16	20	20	24
... stories	20	24	24	28
... stories	24	28	24	28
... stories	28	32	28	32

2. Walls of the Superstructure

Brick and Stone Walls. Very little is known regarding the STABILITY of ... of buildings beyond what has been gained by practical experience. The ... stresses in any horizontal sections of such walls, which can be determined ... any accuracy, are the direct weight of the walls above and the pressure ... the floors and roof. In most walls, however, there is a tendency to ..., to overcome which it is necessary to make them thicker than would ... required to resist the DIRECT CRUSHING STRESS. The resistance to fire should ... taken into account in deciding upon the thickness of any given wall.

The strength of a wall depends also very much upon the quality of the m used and upon the way in which the wall is built. A wall bonded ever in height, and with every joint slushed full with good rich mortar, is as as a poorly built wall 4 in thicker. Walls laid with cement mortar a much stronger than those laid with lime mortar, and a brick wall bui bricks that have been well wet just before laying is very much stronger th built with dry bricks.

Thickness of External Walls. In nearly all the larger cities of the c the minimum thickness of the walls is prescribed by law or ordinance, these requirements are generally ample they are commonly adhered to by tects when designing brick buildings. Table III * gives the thickness o walls required for MERCANTILE BUILDINGS in representative cities of di sections of the United States, and affords about as good a guide as o have, because the values given, as a rule, represent the judgment o qualified and experienced persons. Walls for DWELLINGS are generall mitted to be 4 in less in thickness than for warehouses, although in cities little or no distinction is made between business blocks and dwelli

Table IV gives the thickness required for the brick walls of dwellings ments, hotels and office-buildings † in Chicago. The thickness given is th mum that should be allowed for the walls of such buildings, unless special conditions exist. For modifications for different classes of buildi the building code. In St. Louis the two upper stories of dwellings are r to be 13 in, the next two below, 18 in, the next two 22 in, and the next tw thick.

In compiling Table III the top of the second floor was taken at 19 ft the sidewalk, and the height of the other stories at 13 ft 4 in, includi thickness of the floor, as the New York and Boston laws and the laws o other cities give the height of the walls in feet instead of in stories. W height of stories exceeds these measurements the thickness of the walls i cases will have to be increased. The Chicago ordinance (1916) specifi "where 12-in walls are used, the story-heights shall not exceed 18 ft, whe walls are used, the story-heights shall not exceed 24 ft, and where 20-i are used, the story-heights shall not exceed 30 ft."

General Rule for Thickness of Walls. Although there are great ences in the thickness given in Table III, more indeed than there sho a general rule might be formulated from it, for MERCANTILE BUILDIN four stories in height, which would be somewhat as follows:

For bricks equal to those used in Boston or Chicago, make the thick the three upper stories 16 in, of the next three below 20 in, the next thr and the next three 28 in. For a poorer quality of material make only t upper stories 16 in thick, the next three 20 in, and so on down. In buildi than five stories in height the top story may be 12 in thick.

In determining the thickness of walls the following general principles be recognized:

(1) That walls of warehouses and mercantile buildings should be than those used for living or office purposes.

* Since this table was compiled, some provisions of some laws have been chan the requirements relating to the thicknesses of walls vary but little from thos As building laws of different cities are amended from time to time, architects and must be guided by the code in force in the city in which a building is to be erecte table represents the average requirements and is useful for comparative purpose a guide for those building outside of cities, or where no special building laws are

† For other than steel skeleton construction.

Table III.* Thickness in Inches of Walls for Mercantile Buildings and, Except in Chicago, for All Buildings Over Five Stories in Height

Height and location of building		Stories							
		1st	2d	3d	4th	5th	6th	7th	8th
Two stories	Boston	16	12	....	....	....	....	....	....
	New York	12	12	....	....	....	....	....	....
	Chicago	12	12	....	....	....	....	....	....
	Minneapolis	12	12	....	....	....	....	....	....
	St. Louis	18	13	....	....	....	....	....	....
	Denver	13	13	....	....	....	....	....	....
	San Francisco	17	13	....	....	....	....	....	....
	New Orleans	13	13	....	....	....	....	....	....
Three stories	Boston	20	16	16	....	....	....	....	....
	New York	16	16	12	....	....	....	....	....
	Chicago	16	12	12	....	....	....	....	....
	Minneapolis	16	12	12	....	....	....	....	....
	St. Louis	18	18	13	....	....	....	....	....
	Denver	17	17	13	....	....	....	....	....
	San Francisco	17	17	13	....	....	....	....	....
	New Orleans	13	13	13	....	....	....	....	....
Four stories	Boston	20	16	16	16	....	....	....	....
	New York	16	16	16	12	....	....	....	....
	Chicago	20	16	16	12	....	....	....	....
	Minneapolis	16	16	12	12	....	....	....	....
	St. Louis	22	18	18	13	....	....	....	....
	Denver	21	17	17	13	....	....	....	....
	San Francisco	17	17	17	13	....	....	....	....
	New Orleans	18	18	13	13	....	....	....	....
Five stories	Boston	20	20	20	20	16	....	....	....
	New York	20	16	16	16	16	....	....	....
	Chicago	20	20	16	16	16	....	....	....
	Minneapolis	20	16	16	12	12	....	....	....
	St. Louis	22	22	18	18	13	....	....	....
	Denver	21	21	17	17	13	....	....	....
	San Francisco	21	17	17	17	13	....	....	....
	New Orleans	18	18	18	13	13	....	....	....
Six stories	Boston	24	20	20	20	20	16	....	....
	New York	24	20	20	20	16	16	....	....
	Chicago	20	20	20	16	16	16	....	....
	Minneapolis	20	20	16	16	16	12	....	....
	St. Louis	26	22	22	18	18	13	....	....
	Denver	26	21	21	17	17	13	....	....
	San Francisco	21	21	17	17	17	13	....	....
	New Orleans	22	18	18	18	13	13	....	....
Seven stories	Boston	24	20	20	20	20	20	16	....
	New York	28	24	24	20	20	16	16	....
	Chicago	20	20	20	20	16	16	16	....
	Minneapolis	20	20	20	16	16	16	12	....
	St. Louis	26	26	22	22	18	18	13	....
	Denver	26	21	21	21	17	17	17	....
	New Orleans	22	22	18	18	18	13	13	....
Eight stories	Boston	28	24	20	20	20	20	20	16
	New York	32	28	24	24	20	20	16	16
	Chicago	24	24	20	20	20	16	16	16
	Minneapolis	24	20	20	20	16	16	16	12
	St. Louis	30	26	26	22	22	18	18	13
	Denver	30	26	21	21	21	17	17	17
	New Orleans	22	22	22	18	18	18	13	13

* See paragraphs and foot-note on page 230.

Table III (Continued).* Thickness in Inches of Walls for Mercanti[illegible] Buildings and, Except in Chicago, for all Buildings Over Five Stories in Height

Height and location of building		Stories 1st	2d	3d	4th	5th	6th	7th	8th	9th	10th	11[illegible]
Nine stories	Boston	28	24	24	20	20	20	20	20	16	....	...
	New York	32	32	28	24	24	20	20	16	16	...	...
	Chicago	24	24	24	20	20	20	16	16	16	....	...
	Minneapolis	24	24	20	20	20	16	16	16	12	....	...
	St. Louis	30	30	26	26	22	22	18	18	13	....	...
	Denver	30	26	26	21	21	21	17	17	17	....	...
Ten stories	Boston	28	28	24	24	20	20	20	20	20	16	...
	New York	36	32	32	28	24	24	20	20	16	16	...
	Chicago	28	28	24	24	24	20	20	20	16	16	...
	Minneapolis	24	24	24	20	20	20	16	16	16	12	...
	St. Louis	34	30	30	26	26	22	22	18	18	13	...
	Denver	30	30	26	26	21	21	21	17	17	17	...
Eleven stories	Boston	36	32	32	28	28	24	20	20	20	20	[illegible]
	New York	36	36	32	28	28	24	24	20	20	16	[illegible]
	Chicago	28	28	24	24	24	20	20	20	16	16	[illegible]
	St. Louis	34	34	30	30	26	26	22	22	18	18	[illegible]
	Denver	30	30	26	26	26	21	21	21	17	17	[illegible]
Twelve stories	Boston	36	36	32	32	28	28	24	20	20	20	[illegible]
	New York	40	36	36	32	32	28	24	24	20	20	[illegible]
	Chicago	28	28	28	24	24	24	20	20	20	16	[illegible]
	St. Louis	34	34	34	30	30	26	26	22	22	18	[illegible]
	Denver	30	30	30	26	26	26	21	21	21	17	[illegible]

* See footnote on page 230.

Table IV.† Thickness of Enclosing Walls for Residences, Tenemen[illegible] Hotels and Office-Buildings.‡ Chicago Building Ordinance (1916

Number of stories	Basement	Stories 1st	2d	3d	4th	5th	6th	7th	8th	9th	10th	11[illegible]
Basement and...												
One-story	12	12	...	...	...	...	...	...	...	...	...	.
Two-story	16	12	12	...	...	...	...	...	...	...	...	.
Three-story	16	16	12	12	...	...	...	...	...	...	...	.
Four-story	20	20	16	16	12	...	...	...	...	...	...	.
Five-story	24	20	20	16	16	16	...	...	...	...	...	.
Six-story	24	20	20	20	16	16	16	...	...	...	...	.
Seven-story	24	20	20	20	20	16	16	16	...	...	...	.
Eight-story	24	24	24	20	20	20	16	16	16	...	...	.
Nine-story	28	24	24	24	20	20	20	16	16	16	...	.
Ten-story	28	28	28	24	24	24	20	20	16	16	16	.
Eleven-story	28	28	28	24	24	24	20	20	20	16	16	[illegible]
Twelve-story	32	28	28	28	24	24	24	20	20	20	16	[illegible]

† These thicknesses are allowed when certain requirements are fulfilled in re[illegible] ngths of walls, heights of stories, etc. For these modifying restrictions and [illegible]assifications of buildings in regard to their uses the building laws must be co[illegible] he table is inserted in this form as a useful general guide and as an illustratio[illegible] verage contemporary practice. For modifications for different classes of buildi[illegible] de.

‡ For other than steel skeleton construction.

That high stories and clear spans exceeding 25 ft require thicker

That the length of a wall is a source of weakness, and that the thickness be increased 4 in for every 25 ft over 100 or 125 ft in length. In New the thicknesses given in the table must be increased for buildings exceeding ft in depth on the lot. In Western cities the tables are compiled for 125 ft in depth, as that is the usual depth of lots in those

That walls with over 33% of openings should be increased in thickness. That partition walls may be 4 in less in thickness than the outside walls over 60 ft long, but that no partition should be less than 8 in thick.

Faced with Ashlar. "Bearing walls faced with ashlar shall be at least thick. Ashlar shall not be included in reckoning the thickness of walls it is either at least 8 in thick or alternately 4 in and 8 in to allow at least bond. Ashlar not having at least a 4-in bond in alternate courses must be the backing by metal anchors, one to each block 3 ft or less long, and each block over 3 ft long." *

Walls should generally be 4 in thicker than required for brick

Walls. Hollow walls are undoubtedly desirable for dwellings, and be used for other buildings not more than four or five stories in height, of the security afforded from the weather. Owing to the fact that are usually more expensive than solid walls and occupy more space, they very extensively used in this country.

Boston building law requires that vaulted walls shall contain, exclusive the same amount of material as is required for solid walls, and the on the inside of the air-space in walls over two stories in height shall less than 8 in thick, and the parts on either side shall be securely tied with ties not more than 2 ft apart in each direction.

of Concrete Blocks. Blocks made of Portland-cement concrete, and in molds, are frequently used for building walls and partitions that are thin and bear light loads. Patents have been taken out on forms of blocks and on machines or processes for making the same. buildings have been erected with walls built of these blocks. Most of are molded so as to form hollow walls. Block construction of this an advantage over poured walls, in that the blocks are thoroughly before they are set and hence no provision is required for expansion or For the thin, light walls above mentioned the concrete-block is better adapted than solid concrete. The expense of forms is and also the tendency to crack and to leave an unsatisfactory surface. Concrete blocks may be substituted for any ordinary stone or brick. Building laws usually require the thickness of walls of hollow con- to be not less than that required for brick walls. They should not in party walls. (See, also, Chapter XXIII, Subdivision 2.)

of Hollow Tiles. Hollow tiles are used for the external walls of and sometimes for factories in some locations and under certain For example, the building laws (1913) of the District of Columbia hollow tiles, not less than 12 in in thickness, to be used for the

* Boston Building Law, in force in 1915.

external walls of dwellings located not less than 3 ft from the side or pa of the lot. The Philadelphia laws do not allow the use of hollow tiles external wall or heavy bearing partition. As far as fire-resistance is con construction of hollow tiles is, of course, superior to wooden construction, use is increasing, the outside walls being usually covered with cement or although occasionally left with the finished texture of the tile surface reason hollow tiles are prohibited by building ordinances for certain because when heated and then suddenly cooled by water they are apt t from the sudden contraction. Recent conflagrations have shown tha burned terra-cotta will crack and fall to pieces under severe heat alone. also, Chapter XXIII, Subdivision 2.)

Party Walls. There is much diversity in building regulations re the thickness of party walls, although they all agree in that such walls never be less than 12 in thick. About one-half of the laws require th walls shall be of the same thickness as external walls; the remainder ar equally divided between making the party walls 4 in thicker or thinn for independent side walls. When the walls are proportioned by previously given the author believes that the thickness of the party wall be increased 4 in in each story. The floor-load on party walls is o twice that on side walls, and the necessity for thorough fire-prote greater in the case of party walls than in other walls.

Enclosing Walls for Steel, Skeleton Construction. In building skeleton type the outer masonry walls are usually supported either i story or every other story by the steel framework, and carry nothing b own weight. Such walls may, therefore, be considered as only one stories high, and are usually made only 12 in thick for the whole hei twelve-story or fifteen-story building. For SKELETON CONSTRUCTI Chicago ordinance allows ENCLOSING WALLS of 12-in thickness for all The former New York City code* required the use of 12-in enclosing wal ft of the uppermost height thereof, or to the nearest tier of beams to that ment, and 4 in additional thickness for every lower 60-ft section or to t est tier of beams to such vertical measurement, down to the tier of bea est to the curb-level. But, on account of the severity of some of the ments as applied to very high buildings of skeleton construction, pe was frequently given by the Commissioners of Buildings, who were em to modify the building laws within certain limits, to reduce the thi of certain walls for very high buildings, according to the peculiar circum of each case, without endangering the strength and safety of the A few of the earlier tall buildings were built with SELF-SUSTAINING starting from the foundation, while columns were introduced merely to the floors and to give additional stiffness. "The World Building, N City, erected in 1890, is an extreme example of high-building cons with self-sustaining walls. The main roof is 191 ft above the str making thirteen main stories, above which is a dome containing six s all, a height of 275 ft above the street. The self-sustaining walls are sandstone, brick and terra-cotta, the thickness increasing from 2 ft a to as much as 11 ft 4 in near the bottom, where the walls are offset to a footing 15 ft wide. The walls are vertical on the outside faces, the being varied by inside offsets, so that the columns are recessed into at the bottom, but emerge and are some distance clear of the walls at t

* The revised Code, 1916-17, allows 12-in curtain walls in skeleton buildings height of building, when supported on girders in each story. This practice i by about fifty other cities.

† From Architectural Engineering, by J. K. Freitag.

4. Natural Cements and Mortars*

perties and Uses of Natural Cements. The first hydraulic cements in this country were NATURAL CEMENTS, manufactured by the calcination ...laceous limestone containing sufficient silica, alumina and iron oxide to ... hydraulic properties when the burned rock was pulverized and gauged ... water. These natural cements were very widely manufactured and used ... recent years, when they have been practically completely replaced by ...and cement. Natural cements vary in color from light yellow to dark ... according to the content of oxide of iron, and in distinction to Portland ...ts they are not uniform in their composition or behavior. The chemical ...sition and physical characteristics of various natural cements vary within ...imits, not only between cements manufactured in different mills, but be- ... the products of the same mill at different times. Natural cements set ... rapidly than Portland cements and are slower in developing strength. ...production of natural cement in the United States for 1913 was 800 000 ...s, while during the same year the production of Portland cement was ...000 barrels; from which it is seen that the natural-cement industry is ...ely almost extinct. Natural cement may be used in massive masonry ... weight rather than strength is the essential feature. It is used, also, ...rtain special purposes, such as in the manufacture of safes and in certain ...tries where a quick-setting cement is necessary. Where economy is the ...ing factor, a comparison may be made between the use of natural cement ... leaner mixture of Portland cement that will develop the same strength.

...ight. The specifications of the American Society for Testing Materials ...e that a bag of natural cement shall contain 94 lb, net, of cement, and that ... barrel of natural cement shall contain three bags of this NET WEIGHT.

...ength. A natural-cement mortar, in order to comply with the require- ...s of the standard specifications of the American Society for Testing Ma- ...s, must show a TENSILE STRENGTH, for the neat cement, of at least 150 lb ... in, when one week old, and 250 lb at the end of 28 days; or, when mixed ... three parts of standard Ottawa sand, 50 lb at the end of one week, and 125 ... the end of 28 days. The strength of 1 : 2 natural-cement mortar is about ... to that of 1 : 4 Portland-cement mortar.

...portions of Natural Cement and Sand for Mortar and Concrete. ...ortar for rubble-stone masonwork and ordinary brickwork, one part of ...l cement may be mixed with three parts of sand, by measure.

...draulic Lime. A product closely related to natural cement is HYDRAULIC ... This is manufactured in the same way as natural cement, but the rock ...contains sufficient lime to permit it to slake like quicklime. When the ...ing product is pulverized, it sets and hardens as an hydraulic cement. ...ulic limes are largely manufactured in Europe, and especially in France ...elgium, but in the United States they have been manufactured only in a ...alities. This is due to the fact that while rock of suitable composition ...ly found, the impurities are not uniformly distributed through it, but are ... in layers or seams which prevent the material from being uniformly ...d. The portion of the rock immediately adjacent to and including the ...f impurities overburns, frequently melting like a slag, while the purer ...s consist simply of quicklime; and while the resulting mass slakes partly, ...duct when pulverized is unreliable as a cement.

* ...tical data relating to Cements, Limes and Plasters were furnished the Editor by ...les Warner Company of Wilmington, Del. For Limes and Plasters, see Part ...es 1548 to 1558.

Grappier Cement is a BY-PRODUCT produced during the calcination DRAULIC LIME.

La Farge Cement is an imported NON-STAINING GRAPPIER CEME develops nearly the same strength as the Portland cements.

5. Artificial Cements and Mortars

The Artificial Cements used in the United States include Portland and Puzzolan or slag cement.

Portland Cement. The principal artificial cement in this countr is PORTLAND CEMENT. It is manufactured from two raw materials wl ground to extreme fineness to secure an intimate mix before burning, is from this fact that it derives its name, ARTIFICIAL CEMENT. Th terials must be so proportioned that in the finished cement, silica, alumi oxide and lime will be present in a certain ratio which must be mai within close limits. In the Lehigh Valley region of Pennsylvania, i are located some of the leading Portland-cement mills of the United St raw materials used are limestone and cement-rock. The cement-rock i pure limestone carrying argillaceous or clay-matter. In order to bring t content up to the required percentage, it is usually found necessary region to add limestone. In other districts the raw materials used a stone and clay, limestone and shale, marl and clay and also blast-furn and limestone. The product from the last-mentioned mixture shoulc confused with the common slag cement or Puzzolan cement, as the slag i used as a raw material supplying silica, alumina, iron oxide and lime; a the exception of the use of slag to furnish these ingredients, the process o facture and the properties are substantially the same as for the other I cements. The raw mix in a Portland cement mill is analyzed at mc several times each hour to keep the composition of the cement within th limits. The raw material, which is pulverized as fine as the finished ce burned in rotary kilns, the fuel used in most instances being powder From the kiln it issues in the form of CLINKER, the name given to the rified product. After cooling, calcium sulphate in the form of gy added to control the set and the product is pulverized and packed for sh The manufacture and properties of Portland cement have been made the of careful study by the American Society for Testing Materials and American Society of Civil Engineers. The result of this study is emb the standard specifications of the American Society for Testing Mate tracts from which are given in the paragraphs following. These speci furnish a reliable guide for the acceptance or rejection of any shipmer ment and have been very widely adopted by the leading architects and e of this country. These specifications do not stipulate that Portland cem consist of any one particular composition, but in this respect confine th to the limitation of the magnesia (MgO) and anhydrous sulphuric acid (S tent. The reason for this is that with different raw materials it is foun sary to vary the composition of the cement to obtain the correct properties in the finished material. Different cements which satisfy the ments of these standard specifications are generally considered satisfac ments for use, although the composition of one may vary in some pa from that of another. The CHEMICAL COMPOSITION of a good brand land cement is about as follows: Lime, 62; silica, 23; alumina, 8; and im such as iron oxide, magnesia, and sulphuric acid, 7.

STANDARD SPECIFICATIONS FOR PORTLAND CEMENT.* The following give the most important requirements for Portland cement:

* From the Standard Specifications and Tests for Portland Cement, revised, fective, January 1, 1917), by the American Society for Testing Materials.

(1) DEFINITION. Portland cement is the product obtained by finely pulveriz- clinker produced by calcining to incipient fusion an intimate and properly pro- portioned mixture of argillaceous and calcareous materials, with no additions sub- sequent to calcination excepting water and calcined or uncalcined gypsum. (2) CHEMICAL LIMITS. The following limits shall not be exceeded:

Loss on ignition, per cent	4.00
Insoluble residue, per cent	0.85
Sulphuric anhydride (SO_3), per cent	2.00
Magnesia (MgO), per cent	5.00

(3) SPECIFIC GRAVITY. The specific gravity of cement shall be not less than (3.07 for white Portland cement). Should the test of cement as received fall below this requirement a second test may be made upon an ignited sample. The specific-gravity test will not be made unless specifically ordered. (4) FINENESS. The residue on a standard No. 200 sieve shall not exceed 22 per cent by weight. (5) SOUNDNESS. A pat of neat cement shall remain firm and hard, and show no signs of distortion, cracking, checking, or disintegration in the steam test for soundness. (6) TIME OF SETTING. The cement shall not develop initial set in less than 45 minutes when the Vicat needle is used, or 60 minutes when the Gillmore needle is used. Final set shall be attained within 10 hours. (7) TENSILE STRENGTH. The average tensile strength in pounds per square inch of not less than three standard mortar briquettes composed of 1 part cement and 3 parts standard sand, by weight, shall be equal to or higher than the following:

Age at test, days	Storage of briquettes	Tensile strength, lb per sq in
7	1 day in moist air, 6 days in water	200
28	1 day in moist air, 27 days in water	300

(8) The average tensile strength of standard mortar at 28 days shall be higher than the strength at 7 days. (9) PACKAGES AND MARKING. The cement shall be delivered in suitable bags or barrels with the brand and name of the manu- facturer plainly marked thereon, unless shipped in bulk. A bag shall contain 94 lb net. A barrel shall contain 376 lb net. (10) STORAGE. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment, and in a suitable weather-tight building which will protect the cement from dampness. (11) INSPECTION. Every facility shall be provided the purchaser for careful sampling and inspection at either the mill or at the site of the work, as may be specified by the purchaser. At least 10 days from the time of sampling shall be allowed for the completion of the 7-day test, and at least 31 days shall be allowed for the completion of the 28-day test. The cement shall be tested in accordance with the methods hereinafter prescribed. The 28-day test shall be waived only when specifically so ordered. (12) REJEC- TION. The cement may be rejected if it fails to meet any of the requirements of these specifications.

Sections (13) to (15), also, relate to Rejection. (See complete Specification.)

Puzzolan or Slag Cements are not used extensively and never in important work. Their manufacture and properties may be briefly described as follows: Blast-furnace basic slag is granulated by running it in a molten condition into water. This accomplishes two objects. The slag is broken up into fine particles and the sudden chilling enhances its hydraulic properties. These particles are dried and ground with hydrated lime, in the proportion of from 15 to 25% hydrated lime and from 75 to 85% of granulated slag. Such cement, known as SLAG CEMENT, is slow-setting and slow-hardening, and does not develop as

much strength as natural or Portland cement. Slag cements are characte by their light lilac color, their extreme fineness and their low specific gra They are considered unreliable for use except for foundation-work under gr where they are not exposed to air or running water.

Stainless Cements. Any ordinary Portland or natural cement will limestones, some porous marbles, some granites and some other light-co stones. The best non-staining material is lime, that is, lime free from exce iron oxide. There are some Portland cements, however, which are called STAINING CEMENTS, and where care is used in their manufacture and the free or comparatively free from iron oxide, they cause no trouble. Ar the non-staining cements which have been extensively used for masonr which staining would be objectionable, is La Farge Cement, before mentic It is made at Teil, France, is light-colored and contains a small percenta iron and soluble salts. There are other non-staining cements on the ma For setting stones, and in order to RETARD THE SETTING of the cement unti stones are well bedded, 1 part by volume of lime-paste is usually mixed 4 parts of the cement.

Cost of Portland Cement.* Portland cement can now (1915) be purch in this country at prices ranging from 90 cents to $2.50 per barrel, free on b cars at the mills. The cost of the sacks and the freight are extra. The r price for single barrels varies from about $2.00 to $2.50 per barrel. As a the cost of cement in carload lots is about 85 cts per bbl at the mills. An e charge of 10 cts per bbl for bags is made when the cement is delivered in p bags. The extra charge is 40 cts, if delivered in cloth, but the mills refund 40 cts when the bags are returned in good condition. There is a charge c cts when the cement is furnished in wooden barrels and no allowance is r for barrels returned. It is generally cheaper in the end to buy the ceme cloth bags and return the empty bags. For about 500 miles, the fre charges are about 40 cts per bbl of cement, making the total cost per bb this distance $1.25, when purchased in cloth bags and when the 40 cts per are refunded. Testing costs from 3 to 5 cts per bbl, or from $5 to $6 carload. Unloading and storing near the station cost about 3 cts per bbl, about 2 cts per bbl are usually added to the costs to allow for handling returning empty sacks, and freight-charges for and damage to same. Tea costs about 5 cts per bbl per mile. The total cost, therefore, accordin these average costs, is about $1.38 per bbl for the cement ready for us mortar or concrete. (For Cost of Concrete, see page 249; also foot-note for s

Water Required in Mixing Cement Mortar. Good Portland ce requires relatively little water to make a good mortar. Neat cement will from 20 to 22% (by weight) of water to produce the normal consistenc quick-setting cement requiring more water than one that is slow-setting. greater quantity of water is required, it indicates the presence of an exce free lime. When sand is mixed with cement, in the proportion of 3 to 1, more than from 9 to 12½% (by weight) of water will be required. Na cements and slag cements require more water than do Portland cements. much water drowns the cement, retards the setting and weakens the mo Cements can also be weakened or even spoiled by a deficiency of water.

Portland-Cement Mortar. For first-class mortar not more than 3 b sand should be added to 1 bbl of cement. For rubble stonework under ordi conditions a mortar composed of 4 parts of sand to 1 of cement will answer e purpose, and be much stronger than lime mortar. For the top surface of f

* See foot-note, page 249.

alks, from 1 to 1½ parts of sand may be mixed with 1 part of cement. 3 Portland-cement mortar has about the same strength at the end of one as 1 to 1 natural-cement mortar. Mortar made with fine sand requires a larger quantity of cement to obtain a given strength than mortar made with se sand. (See page 276 for ideal mortar with hydrated lime for brickwork.)

ects of Low Temperatures and Freezing on Cement Mortars. The of setting and hardening of cement mortar is greatly affected by the temper-e, and the exposure and loading of new work often depends upon the pre-ing temperature. The freezing of natural-cement mortars should be en-y avoided as it seriously injures them. Although freezing greatly retards hardening of Portland-cement mortars and concretes, it does not appear injure them. Thin coats of mortar, such as plaster, and troweled surfaces se on which free moisture is formed should not be applied in freezing ther as they are apt to scale. In general, it is undesirable to work with tar or concrete in freezing weather, as the difficulties of properly mixing placing the materials are then increased; it must be admitted, however, t successful work with Portland-cement mortar and concrete has been done emperatures considerably below freezing.

he Effect of Salt in Mortar. When salt is added to the water of mix-g, the freezing-point is lowered, and, within certain limits, the freezing of mortar or concrete is prevented. The ultimate strength of mortar does not ar to be reduced when the amount of salt does not exceed 10%. Tetmajer s the amount of salt required to lower the freezing-temperature as equal to of the weight of the water per degree F. below 32°. The rule for the pro-tion of salt used in the works at Woolwich Arsenal, is said to have been as ws: "Dissolve 1 lb of rock-salt in 18 gal of water when the temperature is 2° F., and add 3 oz of salt for every three degrees of lower temperature."

ffect of Hot Water and of Soda. Hot water hastens the setting of tland-cement mortar, and 2 lb of carbonate of soda in 1 gal of water, boiled mixed in mortar, hastens the setting and lessens the danger of freezing.

uantity of Mortar required for Masonry and Plastering.* "One bbl Portland cement and 3 bbl of sand, thoroughly and properly mixed, will ke 3¼ bbl, or 12 cu ft of good strong mortar. This will be sufficient to lay 1½ cu yd of rough stone, or about 750 bricks, with from ¼ to ⅜-in joints, over 125 sq ft of surface, 1 in thick, or 250 sq ft, ½ in thick."

"One bbl of natural cement and 2 bbl of lime, mixed with about ½ bbl of er, will make 8 cu ft of mortar, sufficient to lay 522 common bricks, with ¼ to ⅜-in joints, or about 1 cu yd of rough rubble."

For the top coat of walks or floors, 1 bbl of Portland cement and 1 of sand cover from 75 to 80 sq ft, ½ in thick, or from 50 to 56 sq ft, ¾ in thick. One bbl of Portland cement and 1½ bbl of sand will cover from 110 to 120 sq f floor, ½ in thick, or from 75 to 80 sq ft, ¾ in thick.

he Mixing of Mortar. Mortar may be mixed by hand or by mechanical ers, the latter being preferable for the mixing of large quantities. When mixing is by hand, it should be done on platforms made water-tight to pre-t the loss of cement. The cement and sand should be mixed dry in small ches and in the proportions required, the platform being clean. Water is ed and the whole mass remixed until it is homogeneous and leaves the mixing clean when drawn out. Mortar should never be retempered after it has n to set.

*These figures can be considered as approximate only, as the amount of mortar will y on different jobs.

Adhesive Strength of Portland Cement, Sulphur and Lead for Anc ing Bolts.* "Fourteen holes were drilled in a ledge of solid limestone, s of them being 1¾ and seven 1⅝ in in diameter, and all being 3½ ft deep. S ¾ and seven 1-in bolts were prepared with thread and nut on one end and the other end plain but ragged for a length of 3½ ft.

"Four were anchored with sulphur, four with lead and six with cem mixed neat. Half the number of the ¾-in and 1-in bolts being thus anchored each of the three materials, all stood until the cement was two weeks Then a lever was rigged and the bolts pulled, with the following results.

"Sulphur: Three bolts out of four developed their full strength 16 000 31 000 lb. One 1-in bolt failed by drawing out, under 12 000 lb. Lead: T bolts out of four developed their full strength, as above. One 1-in pulled out, under 13 000 lb. Cement: Five of the bolts out of six broke w out pulling out. One 1-in bolt began to yield in the cement at 26 000 lb, sustained the load a few seconds before it broke.

"While this experiment demonstrated the superiority of cement, both a strength and ease of application, it did not give the strength per square inc area. To determine this, four specimens of limestone were prepared, 10 in wide, 18 in long and 12 in thick, two of them having 1¾-in holes, and of them 2¾-in holes drilled in them. Into the small holes 1-in bolts w cemented, one of them being perfectly plain round iron, and the other havi thread cut on the portion which was embedded in the cement. Into the 2¾ holes were cemented 2-in bolts similarly treated, and the four specimens w allowed to stand 13 days before completing the experiment. At the end of time they were put into a standard testing-machine and pulled. The plain bolt began to yield at 20 000 lb and the threaded one at 21 000 lb. The plain bolt began to yield at 34 000 lb and the threaded one at 32 000 lb, force in all cases being very slowly applied. The pump was then run at a gre speed, and the stones holding the 2-in bolts split at 67 000 lb in the case of smooth one and at 50 000 lb in the case of the threaded one.

"It is thus seen that for anchoring bolts in stone, cement is more relia stronger and easier of application than either lead or sulphur, and that its sistance is from 400 to 500 lb per sq in of surface exposed. It is also a w ascertained fact that it preserves iron rather than corrodes it. The cem used throughout the experiment was an English Portland cement."

8. Concrete †

Properties and Uses of Concrete.‡ There is probably no material th so enduring or better adapted for foundations, walks and basement floors, than cement concrete, and for certain classes of buildings it is used with vantage for the walls, floors and interior supports. There are now thousand buildings in this and other countries in which all of the structural portions formed of reinforced concrete, and the use of Portland-cement concrete f

* The test of these materials is reported in the American Architect, page 105, xxiv.

† The subject of concrete in general, including plain or mass-concrete and reinfo concrete, is to-day so important, and the available data so vast in amount that only t brief statements of general principles and of the best engineering practice that are the i important for the architect and builder to know can be included in a handbook of kind. For full treatments of the subject, the readers are referred to the numerous re treatises, tests, proceedings of engineering societies, etc.

‡ For reinforced Concrete, see Chapter XXIV; for Concrete Foundations, Chapte for Reinforced-Concrete Factory Construction, Chapter XXV; and for Strength of C crete, Chapter V. See, also, Chapter XXIII, pages 817 and 843.

riety of purposes is rapidly extending, due to the reduced price of d cement, and to a better appreciation and understanding of its proper- merits. Concrete may be defined as an artificial stone, made by cement, water and what is called an aggregate, consisting of small and rticles of sand or screenings and gravel or broken stone; and when with good Portland cement, in proper proportions, it becomes so hard that when pieces of it are broken, the line of fracture often passes the particles of stone, showing that the adhesion of the cement to the is greater than the cohesive strength of the stone itself.

Aggregates.* "Extreme care should be exercised in selecting the aggre- for mortar and concrete, and careful tests made of the materials for the e of determining their qualities and the grading necessary to secure m density or a minimum percentage of voids. A convenient coefficient ity is the ratio of the sum of the volumes of materials contained in a lume to the total unit volume. (See, also, pages 908 and 909.)

Fine Aggregates should consist of sand, crushed stone, or gravel screen- aded from fine to coarse and passing when dry a screen having ¼-in diam it preferably should be of siliceous material, and should be clean, coarse, m dust, soft particles, vegetable loam or other deleterious matter, and re than 6% should pass a sieve having 100 meshes per lin in. Fine aggre- hould always be tested. Fine aggregates should be of such quality ortar composed of one part Portland cement and three parts fine aggre- y weight when made into briquettes will show a tensile strength at least to the strength of 1 : 3 mortar of the same consistency made with the ement and standard Ottawa sand. This is a natural sand obtained at a, Ill., passing a screen having 20 meshes and retained on a screen having hes per lin in. It is prepared and furnished by the Ottawa Silica Com- for 2 cts per lb, free on board cars, at Ottawa, Ill., under the direction of ecial Committee on Uniform Tests of Cement of the American Society l Engineers. If the aggregate be of poorer quality the proportion of t should be increased in the mortar to secure the desired strength. If ngth developed by the aggregate in the 1 : 3 mortar is less than 70% of ngth of the Ottawa-sand mortar, the material should be rejected. To he removal of any coating on the grains, which may affect the strength, nds should not be dried before being made into mortar, but should con- tural moisture. The percentage of moisture may be determined upon a e sample for correcting weight. From 10 to 40% more water may be d in mixing bank or artificial sands than for standard Ottawa sand to e the same consistency.

Coarse Aggregates should consist of crushed stone or gravel which is d on a screen having ¼-in diam holes and graded from the smallest to est particles; they should be clean, hard, durable and free from all ous matter. Aggregates containing dust and soft, flat or elongated s should be excluded from important structures."

kind of stone is suitable for the coarse aggregate which has such strength e strength of the concrete is not limited by the strength of the stone. trength is of little advantage beyond this minimum. The stones gener- ployed are granites, traps and limestones. Shales and sandstones of

* of the matter of this paragraph, and of following paragraphs relating to concrete, of data and conclusions formulated by the joint committees of the Am. Soc. C. E., for Test. Mats., Am. Ry. Eng. and Maint. of Way Asso., and Asso. of Am. Cement Manfrs. In regard to Aggregates, etc., see, also, the same subjects in XXIV, pages 908 and 909, and foot-notes on page 908 in that chapter.

deficient strength should be tested before use. Screened gravel generally a good coarse aggregate. "The maximum size of the coarse aggregate is g by the character of the construction. For reinforced concrete and fo masses of unreinforced concrete, the aggregate must be small enough to with the mortar a homogeneous concrete of viscous consistency which readily between and easily surround the reinforcement and fill all parts forms. For concrete in large masses the size of the coarse aggregate increased, as a large aggregate produces a stronger concrete than a fi although it should be noted that the danger of separation from the becomes greater as the size of the coarse aggregate increases."

The use to be made of the concrete determines the maximum size of th aggregate. When used in mass-concrete construction, such as heavy the maximum size may run up to 2½ and 3 in with good results. For rei work and thin walls, however, it is necessary to reduce the maximum s in or less. It has been found that the following are the maximum sizes coarse aggregate of plain or mass-concrete in the best practice: for foun 2½ in; for abutments, 2 in; for arch-rings, 1¼ in; and for copings, thi etc., 1 in.

"Cinder concrete should not be used for reinforced-concrete structu may be allowable in mass for very light loads or for fire-protection p The cinders used should be composed of hard, clean, vitreous clinkers, fi sulphides, unburned coal, or ashes. (See, also, page 909.)

"**Water for Mixing Concrete.** The water used in mixing concrete be free from oil, acid, alkalies, or organic matter."

Preparing and Placing Mortar and Concrete. "(1) **Proportions** materials to be used in concrete should be carefully selected, of uniform and proportioned with a view to securing as nearly as possible a m density.

"(a) Unit of Measure. The unit of measure should be the cubic f bag of cement, containing 94 lb, net, should be considered the equival cu ft. The measurement of the fine and coarse aggregates should be volume.

"(b) Relation of Fine and Coarse Aggregates. The fine and coars gates should be used in such relative proportions as will insure maxim sity. In unimportant work it is sufficient to do this by individual ju using correspondingly higher proportions of cement; for important wo proportions should be carefully determined by density-experiments sizing of the fine and coarse aggregates should be uniformly maintaine proportions changed to meet the varying sizes.

"(c) Relation of Cement and Aggregates. For reinforced-concr struction, one part of cement to a total of six parts of fine and coars gates, measured separately, should generally be used. For column mixtures are generally preferable, and in massive masonry or rubble a mixture of 1 : 9 or even 1 : 12 may be used. These proportions sh determined by the strength or the wearing-qualities required in the c tion at the critical period of its use. Experienced judgment based on in observation and tests of similar conditions in similar localities is an guide as to the proper proportions for any particular case. For all in construction, advance tests should be made of concrete, of the materi portions and consistency to be used in the work. These tests should under laboratory conditions to obtain uniformity in mixing, proportio

* See, also, in Chapter XXIV, paragraphs relating to these subjects on and foot-note relating to the same, on page 908 of that chapter.

and in case the results do not conform to the requirements of the work, [...]s of a better quality should be chosen or richer proportions used to [...] the desired results."

[...] Turneaure of the University of Wisconsin gives the following as [pro]portions of cement, sand and coarse aggregate generally used for various [kinds] of work:

[Re]inforced columns and structural parts [req]uiring extra strength..................	from 1 : 1 : 2 to 1 : 1½ : 3
[For] buildings, thin walls, reinforced concrete, [...]s and impervious construction.........	from 1 : 2 : 4 to 1 : 2½ : 4½
[For] structures requiring great strength rather [tha]n mass..............................	from 1 : 2½ : 5 to 1 : 3 : 6
[For] structures requiring mass rather than [str]ength, foundations, etc..................	from 1 : 3 : 6 to 1 : 4 : 8

[...] **Mixing Concrete.** The ingredients of concrete should be thoroughly [mixed] and the mixing should continue until the cement is uniformly distributed [and the] mass is uniform in color and homogeneous. As the maximum density [and gr]eatest strength of a given mixture depend largely on thorough and com[plete mi]xing, it is essential that the work of mixing should receive special atten[tion an]d care. Inasmuch as it is difficult to determine, by visual inspection, [whethe]r the concrete is uniformly mixed, especially where limestone or aggre[gates ha]ving the color of cement are used, it is essential that the mixing should [continue for] a definite period of time. The minimum time will depend on whether [the mix]ing is done by machine or hand.

[...] **Measuring Ingredients.** Methods of measurement of the proportions [of the] various ingredients should be used which will secure separate and uni[form] measurements of cement, fine aggregate, coarse aggregate and water at [all ti]mes.

[...] **Machine-Mixing.** When the conditions will permit, a machine-mixer [of a ty]pe which insures the uniform proportioning of the materials throughout [the ma]ss should be used, as a more uniform consistency can be thus obtained. [The m]ixing should continue for a minimum time of at least one minute after [all the] ingredients are assembled in the mixer.

[...] **Hand-Mixing.** When it is necessary to mix by hand, the mixing should [be on] a water-tight platform and especial precautions should be taken to turn [the] ingredients together at least six times and until they are homogeneous in [appear]ance and color."

[Th]e most satisfactory method * of mixing concrete by hand is to first prepare [for] mixing of the materials, a tight floor of planks, or, better still, of sheet [iron wi]th the edges turned up about 2 in. Upon this platform should first be [spread] the sand, and upon this the cement. The two should then be thoroughly [and im]mediately mixed by means of shovels or hoes until of an even color. [The] water should be added to make a thin mortar which is then spread [...] The gravel, if used, should then be added, and then the broken stone. [The] stone should be first thoroughly wet, if originally dry. The mass [should] be turned until all the ingredients are thoroughly incorporated and all [the ston]e and gravel covered with mortar, this requiring from four to six turn[ings].

[...] **Consistency.** The materials should be mixed wet enough to result in a [concrete] of such a consistency that it will flow into the forms and about the metal [reinfor]cement when used, and which, at the same time, can be conveyed from

* This paragraph is condensed from several recent specifications.

the mixer to the forms without separation of the coarse aggregate fi mortar.

"(e) Retempering. Mortar or concrete should not be remixed wit after it has partly set."

(3) **Placing Concrete.** "(a) Methods. Concrete after the compl the mixing should be handled rapidly, and in as small masses as is pra from the place of mixing to the place of final deposit, and under no stances should concrete be used that has partly set. A slow-setting should be used when a long time is likely to occur between mixing and Concrete should be deposited in such a manner as will permit the most t compacting, such as can be obtained by working with a straight shovel o tool kept moving up and down until all the ingredients have settled proper places by gravity and the surplus water has been forced to the Special care should be exercised to prevent the formation of LAITANCE, hardens very slowly and forms a poor surface on which to deposit fresh c All LAITANCE should be removed. When suspended work is resum crete previously placed should be roughened, thoroughly cleansed of material and laitance, thoroughly wetted and then slushed with a mor sisting of one part Portland cement and not more than two parts fine ag The faces of concrete exposed to premature drying should be kept w period of at least seven days."

"(b) Mixing and Depositing Concrete in Freezing Weather. (should not be mixed or deposited at a freezing temperature, unless spe cautions are taken to avoid the use of materials covered with ice-cr containing frost, and to provide means to prevent the concrete from after being placed in position and until it has thoroughly hardened. coarse aggregate forms the greater portion of the concrete, it is par important that this material be heated to well above the freezing-point.

"(c) Rubble Concrete. Where the concrete is to be deposited in work, its value may be improved and its cost materially reduced by th clean stones thoroughly embedded in the concrete and as near togetl possible while still entirely surrounded by concrete.

"(d) Depositing Concrete Under Water. In placing concrete under is essential to maintain still water at the place of deposit. The use of T properly designed and operated, is a satisfactory method of placing through water. The concrete should be mixed very wet (more so tha dinarily permissible) so that it will flow readily through the tremies the places with practically a level surface. The coarse aggregate sh smaller than ordinarily used, and never more than 1 in in diameter. Tl gravel facilitates mixing and assists the flow of concrete through the The mouth of the tremie should be buried in the concrete so that it times entirely sealed and the surrounding water prevented from forci into the tremie; the concrete will then discharge without coming in with the water. The tremie should be suspended so that it can be quickly when it is necessary either to choke off or prevent a too rapid f

* Laitance is a whitish, gelatinous substance of about the same composition but with little tendency to harden. It accompanies a disintegration of so cement from the surface of concrete which is exposed to the action of water i is deposited. The concrete is thus weakened and the laitance, also, weakens between old and new material and should be removed before fresh concrete is

† A tremie is a round or square box or tube of wood or plate iron open at th bottom. The diameter varies from 12 to 24 in. The tremie rests in the depo crete, extends above the water-level and is kept full of concrete, which escap bottom as the tube is shifted over the surface.

[illegible] flow should preferably be not over 15 ft. The flow should be continuous [illegible] to produce a monolithic mass and to prevent the formation of laitance [illegible] interior. In large structures it may be necessary to divide the mass of [illegible] into several small compartments or units, filling one at a time. With [illegible] care it is possible in this manner to obtain as good results under water as [illegible] air."

[illegible] for Concrete. "Forms should be substantial and unyielding, so that [illegible] concrete will conform to the designed dimensions and contours, and should [illegible] tight in order to prevent the leakage of mortar. The time for removal of forms [illegible] of the most important considerations in the erection of a structure of con[illegible] or reinforced concrete. Care should be taken to inspect the concrete and [illegible] its hardness before removing the forms. So many conditions affect [illegible] hardening of concrete, that the proper time for the removal of the forms [illegible] be decided by some competent and responsible person, especially where [illegible] atmospheric conditions are unfavorable. It may be stated, in a general [illegible] that forms should remain in place longer for reinforced concrete than for [illegible] massive concrete, and that the forms for floors, beams and similar hori[illegible] structures should remain in place much longer than for vertical walls. [illegible] the concrete gives a distinctive ring under the blow of a hammer, it is [illegible] an indication that it has hardened sufficiently to permit the removal [illegible] forms with safety. If, however, the temperature is such that there is [illegible] possibility that the concrete is frozen, this test is not a safe reliance, as [illegible] concrete may appear to be very hard."

[illegible] Shrinkage of Concrete and Temperature-Changes. "Shrinkage of con[illegible], due to hardening and contraction from temperature-changes, causes [illegible], the size of which depends on the extent of the mass. The resulting [illegible] are important in monolithic construction and should be considered care[illegible] by the designer; they cannot be counteracted successfully, but the effects [illegible] minimized. Large cracks produced by quick hardening or wide ranges [illegible] temperature can be broken up to some extent into small cracks by placing [illegible] in the concrete; in long continuous lengths of concrete, it is [illegible] to provide shrinkage-joints at points in the structure where they will do [illegible] no harm. Reinforcement is of assistance and permits longer distances [illegible] shrinkage-joints than when no reinforcement is used. Small masses or [illegible] bodies of concrete should not be joined to larger or thicker masses without [illegible] for shrinkage at such points. Fillets similar to those used in metal [illegible], but of larger dimensions, for gradually reducing from the thicker to [illegible] thinner body, are of advantage. Shrinkage-cracks are likely to occur at [illegible] where fresh concrete is joined to that which is set, and hence in placing [illegible] concrete, construction-joints should be made on horizontal and vertical [illegible] and, if possible, at points where joints would naturally occur in dimen[illegible] masonry."

[illegible] Effect of Heat on Concrete Fireproofing.* "The actual fire-tests of [illegible] and reinforced concrete have been limited, but experience, together [illegible] the results of tests thus far made, indicates that concrete, on account of [illegible] rate of heat-conductivity and the fact that it is incombustible, may be [illegible] safely for fireproofing purposes. The dehydration of concrete probably [illegible] at about 500° F. and is completed at about 900° F.; but experience indi[illegible] that the volatilization of the water absorbs heat from the surrounding mass, [illegible] together with the resistance of the air-cells, tends to increase the heat-[illegible] of the concrete, so that the process of dehydration is very much re-

* See, also, Chapter XXIII, page 817.

tarded. The concrete that is actually affected by fire remains in positi affords protection to the concrete beneath it. The thickness of the pro coating required depends on the probable duration of a fire which is lil occur in the structure and should be based on the rate of heat-condu The question of the conductivity of concrete is one which requires furthe and investigation before a definite rate for different classes of concrete fully established. However, for ordinary conditions it is recommende the metal in girders and columns be protected by a minimum of 2 in crete; that the metal in beams be protected by a minimum of 1½ in of co and the metal in floor-slabs be protected by a minimum of 1 in of co It is recommended that in monolithic concrete columns, the concrete to a of 1½ in be considered as protective covering and not included in the ef section. It is recommended that the corners of columns, girders and be beveled or rounded, as a sharp corner is more seriously affected by fire round one."

Waterproofing Concrete. "Many expedients have been used to concrete impervious to water under normal conditions, and also under pr conditions that exist in reservoirs, dams and conduits of various kinds. rience shows, however, that where mortar or concrete is proportioned to the greatest practicable density and is mixed to a rather wet consisten resulting mortar or concrete is impervious under moderate pressure. crete of dry consistency is more or less pervious to water, and compou various kinds have been mixed with the concrete, or applied as a wash surface for the purpose of making it water-tight. Many of these com are of but temporary value, and in time lose their power of imparting im ability to the concrete. In the case of subways, long retaining-walls and voirs, provided the concrete itself is impervious, cracks may be so redu horizontal and vertical reinforcement properly proportioned and locate they are too minute to permit leakage or are soon closed by infiltration Coal-tar preparations applied either as a mastic or as a coating on felt or fabric are used for waterproofing, and should be proof against injury by or gases. For retaining-walls and similar walls in direct contact wi earth, the application of one or two coatings of hot coal-tar pitch to th oughly dried surface of concrete is an efficient method of preventing the tration of moisture from the earth." (See, also, Waterproofing for F tions, Part III.

Surface-Finish of Concrete. "Concrete is a material of an individu and should not be used in imitation of other structural materials. One important problems connected with its use is the character of the fi exposed surfaces. The finish of the surface should be determined befo concrete is placed, and the work conducted so as to make possible the desired. For many forms of construction the natural surface of the co is unobjectionable; but frequently the marks of the boards and the fla surface are displeasing, thus making some special treatment desira treatment of the surface either by scrubbing it while green or by tooling it is hard, which removes the film of mortar and brings the aggregates concrete into relief, is frequently used to remove the form-markings, bre monotonous appearance of the surface, and make it more pleasing. The p ing of surfaces should be avoided, for even if carefully done, the plaster is to peel off under the action of frost or temperature-changes."

Design of Massive Concrete. "In the design of massive or plain co no account should be taken of the tensile strength of the material, and s should usually be proportioned, so as to avoid tensile stresses, except in

...ts, to resist indirect stresses. This will generally be accomplished, in ...e of rectangular shapes, if the line of pressure is kept within the middle ...of the section, but in very large structures, such as high masonry dams, a ...exact analysis may be required. Structures of massive concrete are able ...ist unbalanced lateral forces by reason of their weight; hence the element ...ight rather than strength often determines the design. A relatively cheap ...weak concrete, therefore, will often be suitable for massive concrete struc... It is desirable generally to provide joints at intervals to localize the ...t of contraction. Massive concrete is suitable for dams, retaining-walls, ...piers and short columns in which the ratio of length to least width is rela...y small. Under ordinary conditions this ratio should not exceed six. It ...is suitable for arches of moderate span, where the conditions as to founda...s are favorable."

...uantities of Materials Required per Cubic Yard of Concrete.* The ...wing tables give the quantities of Portland cement required to make 1 cu yd ...ortar and the quantities of cement, sand and stone required to make 1 cu ...of concrete. They are based upon formulas deduced by Halbert P. Gillette.

Barrels of Portland Cement per Cubic Yard of Mortar

Voids in sand, 35%, 1 bbl of cement yielding 3.65 cu ft of cement paste

Proportion of cement to sand	1 to 1	1 to 1½	1 to 2	1 to 2½	1 to 3	1 to 4
	bbl	bbl	bbl	bbl	bbl	bbl
...el specified to be 3.5 cu ft	4.22	3.49	2.97	2.57	2.28	1.76
...el specified to be 3.8 cu ft	4.09	3.33	2.81	2.45	2.16	1.62
...el specified to be 4.0 cu ft	4.00	3.24	2.73	2.36	2.08	1.54
...el specified to be 4.4 cu ft	3.81	3.07	2.57	2.27	2.00	1.40
...bic yard of sand per cu yd of mortar..............	0.6	0.7	0.8	0.9	1.0	1.0

Barrels of Portland Cement per Cubic Yard of Mortar

Voids in sand, 45%, 1 bbl of cement yielding 3.4 cu ft of cement paste

Proportion of cement to sand	1 to 1	1 to 1½	1 to 2	1 to 2½	1 to 3	1 to 4
	bbl	bbl	bbl	bbl	bbl	bbl
...el specified to be 3.5 cu ft	4.62	3.80	3.25	2.84	2.35	1.76
...el specified to be 3.8 cu ft	4.32	3.61	3.10	2.72	2.16	1.62
...el specified to be 4.0 cu ft	4.19	3.46	3.00	2.64	2.05	1.54
...el specified to be 4.4 cu ft	3.94	3.34	2.90	2.57	1.86	1.40
...bic yard of sand per cu yd of mortar..............	0.6	0.8	0 9	1.0	1.0	1.0

...n using these tables remember that the proportion of cement to sand is ...ume and not by weight. If the specifications state that a barrel of cement ...be considered to hold 4 cu ft, for example, and that the mortar shall be

*...uoted, by permission, from the Handbook of Cost Data for Contractors and En...., by Halbert P. Gillette, published by The Myron C. Clark Publishing Company, ..., Ill. See 1914 revised edition, pages 538 to 540. This handbook contains com...nd voluminous data on quantities, costs, etc., of building materials and operations.

1 part cement to 2 parts sand, then 1 bbl of cement is mixed with 8 cu ft of regardless of what is the actual size of the barrel, and regardless of how cement paste can be made with a barrel of cement. If the specifications state what the size of a barrel will be, then the contractor is left to guess.

"If the specifications call for proportions by weight, assume a Portlar ment barrel to contain 380 lb of cement, and test the actual weight of a foot of the sand to be used. Sand varies extremely in weight, due both variation in the per cent of voids, and to the variation in the kind of mi of which the sand is composed. A quartz sand having 35% voids weighs per cu ft; but a quartz sand having 45% voids weighs only 91 lb per cu the weight of the sand must be guessed at, assume 100 lb per cu ft. specifications require a mixture of 1 part of cement to 2 parts of sand, by w we will have 380 lb (or 1 bbl) of cement mixed with 2 times 380, or 760 sand; and if the sand weighs 90 lb per cu ft, we shall have 760 divided by 8.44 cu ft of sand to every barrel of cement. In order to use the tables given, we may specify our own size of barrel; let us say 4 cu ft; then, 8.44 di by 4 gives 2.11 parts of sand by volume to 1 part of cement. Without ma error we may call this a 1 to 2 mortar, and use the tables, remembering th barrel is now 'specified to be' 4 cu ft. If we have a brand of cement that 3.4 cu ft of paste per bbl and sand having 45% voids, we find that approxin 3 bbl of cement per cu yd of mortar will be required.

"It should be evident from the foregoing discussions that no table c made, and no rule can be formulated that will yield accurate results unle brand of cement is tested and the percentage of voids in the sand determ This being so, the sensible plan is to use the tables merely as a rough and, where the quantity of cement to be used is very large, to make a few b of mortar, using the available brands of cement and sand in the propo specified. Ten dollars spent in this way may save a thousand, even on a paratively small job, by showing what cement and sand to select."

Ingredients in One Cubic Yard of Concrete *

Sand-voids, 40%; stone-voids, 45%; Portland-cement barrel yielding 3.65 cu paste. Barrel specified to be 3.8 cu ft

Proportions by volume	1 : 2 : 4	1 : 2 : 5	1 : 2 : 6	1 : 2½ : 5	1 : 2½ : 6	1 :
Barrels cement per cu yd concrete	1.46	1.30	1.18	1.13	1.00	1
Cubic yard sand per cu yd concrete	0.41	0.36	0.33	0.40	0.35	0
Cubic yard stone per cu yd concrete	0.82	0.90	1.00	0.80	0.84	0
Proportions by volume	**1 : 3 : 5**	**1 : 3 : 6**	**1 : 3 : 7**	**1 : 4 : 7**	**1 : 4 : 8**	**1 :**
Barrels cement per cu yd concrete	1.13	1.05	0.96	0.82	0.77	c
Cubic yard sand per cu yd concrete	0.48	0.44	0.40	0.46	0.43	c
Cubic yard stone per cu yd concrete	0.80	0.88	0.93	0.80	0.86	c

* This table is to be used where cement is measured packed in the barrel, ordinary barrel holds 3.8 cu ft.

...ill be seen that the above table can be condensed into the following: ...le. Add together the number of parts and divide this sum into ten, the ...t will be, approximately, the number of barrels of cement per cubic yard. ...us for a 1 : 2 : 5 concrete, the sum of the parts is 1 plus 2 plus 5, which is ...en 10 divided by 8 is 1.25 bbl, which is approximately equal to the 1.30 bbl ...in the table. Neither this rule nor this table is applicable if a different ...f cement-barrel is specified, or if the voids in the sand or stone differ mate- ...y from 40% and 45% respectively. There are such innumerable com- ...ons of varying voids, and varying sizes of barrels, that the author does ...em it worth while to give other tables."

Ingredients in One Cubic Yard of Concrete *

...d-voids, 40%; stone-voids, 45%; Portland-cement barrel yielding 3.65 cu ft of paste. Barrel specified to be 4.4 cu ft

...oportions by volume	1 : 2 : 4	1 : 2 : 5	1 : 2 : 6	1 : 2½ : 5	1 : 2½ : 6	1 : 3 : 4
...els cement per cu yd ...ncrete	1.30	1.16	1.00	1.07	0.96	1.08
...ic yard sand per cu yd ...ncrete	0.42	0.38	0.33	0.44	0.40	0.53
...ic yard stone per cu yd ...ncrete	0.84	0.95	1.00	0.88	0.95	0.71

...oportions by volume	1 : 3 : 5	1 : 3 : 6	1 : 3 : 7	1 : 4 : 7	1 : 4 : 8	1 : 4 : 9
...els cement per cu yd ...ncrete	0.96	0.90	0.82	0.75	0.68	0.64
...ic yard sand per cu yd ...ncrete	0.47	0.44	0.40	0.49	0.44	0.42
...ic yard stone per cu yd ...ncrete	0.78	0.88	0.93	0.86	0.88	0.95

...st of Concrete.† (For Cost of Cement, see page 238.) The average ...f sand may be taken at 30 cts per cu yd to cover digging and loading, but ...washed or screened the cost averages between 40 and 55 cts per cu yd. ...ng and freight-charges generally raise the cost of sand, ready to unload at ...e, to from 90 cts to $1.10 per cu yd, and about 15 cts per yd additional ...be added, if unloaded from cars. Gravel costs from $1.20 to $1.40 per cu ...loaded at the job, and crushed stone from $1.45 to $1.60. These prices ...f course, average prices only, and include moderate-haul teaming and un- ...g. For hand-mixing and placing of soft concrete, and spreading without ...mming, the labor-cost varies from 90 cts to $1.30 per cu yd. This is ...ling in barrows materials that are conveniently at hand. This cost ...e much higher for dry concrete, and hand-mixing costs may reach $2 or ...r cu yd. For machine-mixing alone and with machines taking four bags ...batch, the cost of mixing may be even as low as 50 or 60 cts per cu yd. ...cing alone, the cost is about 75 cts per cu yd; this includes wheeling the ...te, dumping it in place and spreading and spading it into forms. This ...ld be almost doubled where unusual care had to be exercised to obtain ...surface and where there was an extra amount of spading. The costs

* ...s table is to be used when the cement is measured loose, after dumping it into ...r under such conditions a barrel of cement yields 4.4 cu ft of loose cement.

† ...onditions changed many costs. Values given are retained temporarily for pur- ...f comparison.

are reduced for heavy mass-concrete, and have been as low as 50 or 60 cu yd for machine-mixing and placing together, by mixer and derrick tracks and cars. The following approximate schedule * of labor-costs fc ing and placing concrete is given by L. H. Allen of the Aberthaw Coı tion Company, in Professor Hool's excellent treatise:

For footings	\$1.50 per cu y
For floor-slabs not exceeding 4½ in in thickness	\$1.60 per cu y
For floor-slabs exceeding 5 in in thickness	\$1.00 per cu y
For columns and thin walls	\$1.50 per cu y
For walls exceeding 18 in in thickness	\$1.00 per cu y
For dams and thick retaining-walls	\$0.70 per cu y

For the unit cost due to the cost of the tools, plant and supplies, \$1 r taken as an average for jobs requiring from 4 000 to 10 000 cu yd of co It varies, of course, with the character and magnitude of the work. Tl for this item is reduced in larger jobs, falling to 80 or even 70 cts per and it is increased in operations of less magnitude to from \$1 to \$1.50 per for, say, 3 000 cu yd of concrete. When the amount of concrete require small as 600 or 700 cu yd, hand-mixing is generally more economica machine-mixing. Mr. Allen summarizes * the cost of 1 cu yd of concret building requiring 5 000 cu yd of reinforced-concrete work in floors and c as follows, the cost of forms and steel and finishing of the surface not included:

Cement, 1¾ bbl, at \$1.38 per bbl	\$2.
Sand, ½ cu yd, at \$1 per cu yd	0.
Stone, 1.35 tons, at \$1.40 per ton	1.
Labor, per cu yd	1.
Plant, per cu yd	1.
Total, per cu yd	\$7.

In this summary the exact theoretical proportions or quantities of sand and stone required for 1 cu yd of concrete, and deduced from fo are not adhered to, the author stating that the exact theoretical proporti the net quantities of the materials determined by careful experimen "conditions on actual construction work do not approach those of lab work and that there is always a considerable waste of cement, sand and In view of these facts, he states that, "when estimating quantities, it is to allow less than the following amounts of cement for different prop of mix:

1 : 1½ : 3 mix	2.00 bbl per cu yd
1 : 2 : 4 mix	1.66 bbl per cu yd
1 : 2½ : 5 mix	1.40 bbl per cu yd
1 : 3 : 6 mix	1.20 bbl per cu yd

It is customary to allow ½ cu yd of sand and 1 cu yd of crushed s 1 cu yd of concrete, and to estimate the weight of crushed stone at 10 cu ft.

The Weight of Concrete varies from 110 to 155 lb per cu ft, acco the material used. Concrete of the usual proportions weighs from 140 t per cu ft. Trap-rock concrete weighs from 148 to 155; limestone o concrete, from 142 to 148; and cinder concrete from 80 to 115 lb per cu

* Reinforced Concrete Construction, by George A. Hool, McGraw-Hill Book C New York.

he Strength of Concrete. See Chapter V.

lier Examples of Portland-Cement Concrete. From the foregoing it n that for foundation-work to-day, mass-concrete varies in proportions a 1 : 3 : 6 to a 1 : 4 : 8 mix. Some of the earlier examples are added for parison.

undations of the United States Naval Observatory, Georgetown, D. C.: nt cement, 2½ sand, 3 gravel, 5 broken stone. (1 bbl of cement, 380 lb, le 1.18 yd of concrete.)

undations of the Cathedral of St. John the Divine, New York: 1 part and cement, 2 parts sand, 3 parts quartz gravel of pieces from 1½ to 2 in meter. (17 000 bbl of cement made 11 000 yd of concrete.)

lanhattan Life Insurance Building, New York, filling of caissons: 1 part n Portland cement, 2 parts sand, 4 parts broken stone.

leston Building (15 stories), New York, filling of caissons: 1 part Portland nt, 3 parts sand, 7 parts stone, finished on top for brickwork with 1 part nt and 3 parts gravel.

ofessor Baker states that the concrete foundations under the Washington ument were made of 1 part Portland cement, 2 parts sand, 3 parts gravel 4 parts broken stone, and that this mixture stood, when six months old, a l of 2 000 lb per sq in, or 144 tons per sq ft.

CHAPTER IV

RETAINING-WALLS, BREAST-WALLS AND VAULT-WALLS

By

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1. Mechanical Principles Involved

General Principles. Before discussing more in detail the problems r to masonry structures, in which, if improperly constructed, a tendency t or overturn on their bases may be developed, a familiarity with what are as the THEOREM OF FRICTION and the THEOREM OF THE MIDDLE THIRD of assistance in comprehending the methods indicated for rendering such tures stable.

Theorem of Friction. If a body rests on an inclined plane it will stationary until the angle ϕ, that the plane makes with the hori

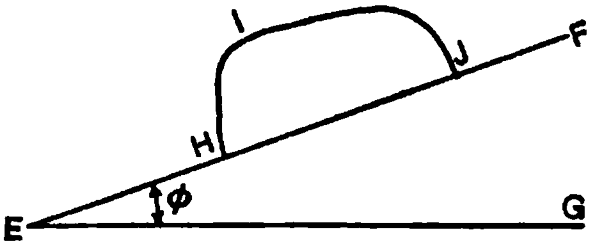

Fig. 1

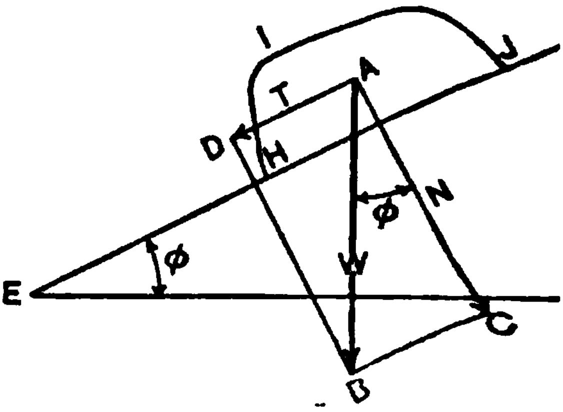

Fig. 2

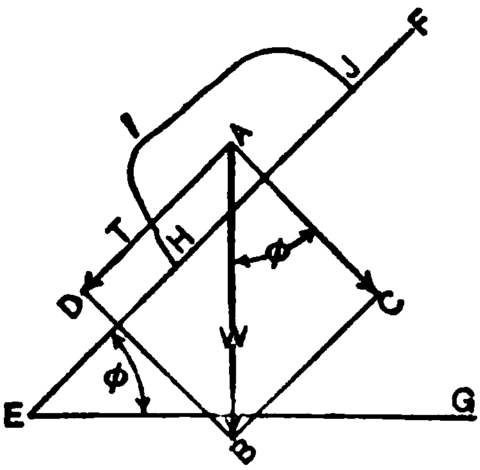

Fig. 3

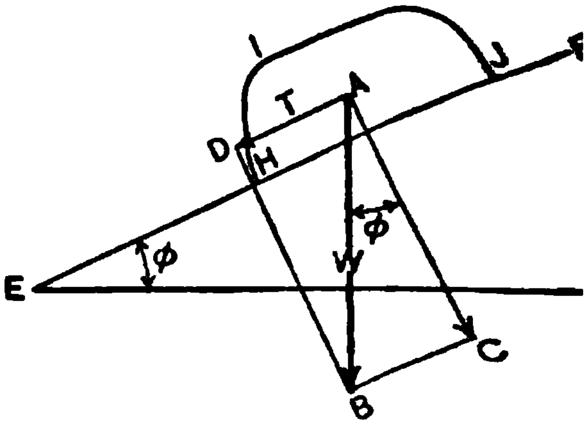

Fig. 4

Figs. 1, 2, 3 and 4. Body on Inclined Plane. Graphical Representation of F

becomes so great that the FRICTION developed between the surfaces of tl and the plane is no longer sufficient to prevent the body from slidin the plane (Fig. 1).

Assume the body HIJ resting on the plane EF. The weight, W, of th is shown graphically by the line AB, applied at its center of gravity A (

ight can be resolved into two component forces, one, AC, normal to the plane and the other, AD, parallel to it. It is the parallel or tangential hich tends to pull the body down the plane and which is resisted by the developed between the two surfaces. The friction developed between e surfaces in contact depends upon the nature of the materials of which e composed and the intensity of the forces pressing them together; and ts the tendency to slide only up to a certain point. As the angle ϕ, the inclined plane makes with the horizontal, increases, the tangential ent T, of the weight W, increases, until it becomes greater than the al resistance, and the body moves down the plane (Fig. 3). From metry,

$$T = W \sin\phi$$

$$N = W \cos\phi, \quad \text{or,} \quad T = N \tan\phi$$

e is evidently a position of the plane, intermediate between the positions in Figs. 1 and 3, in which the component force T is just balanced by the and in which the body remains at rest although just on the point of (Fig. 4). If the angle which the inclined plane makes with the hori-, at the moment when the body is just about to slide, be designated by ϕ, tion developed between the two surfaces will be equal to $N \tan\phi$, since, the angle of inclination of the plane to the horizontal is ϕ, the tangential ent of the weight just balances the friction. From the equation $T = N$ it is evident that the friction is directly proportional to N and to $\tan\phi$. is then known as the COEFFICIENT OF FRICTION and ϕ as the ANGLE OF t, or, in the case of stone surfaces, it is often known as the ANGLE OF se.

following Table I gives the average values of these constants as deter-by experiment.

Table I. Coefficients and Angles of Friction

Kind of surface	Coefficient of friction, tan ϕ	Angle of friction, ϕ
te, limestone and marble:		
dressed upon soft dressed	0.70	35° 00′
d dressed upon hard dressed	0.55	28 50
d dressed upon soft dressed	0.65	33 00
, brick or concrete:		
onry upon masonry	0.65	33 00
onry upon wood (with the grain)	0.60	31 00
onry upon wood (across the grain)	0.50	26 40
onry upon dry clay	0.50	26 40
onry upon wet or moist clay	0.33	18 20
onry upon sand	0.40	21 50
onry upon gravel	0.60	31 00
one upon steel or iron	0.40	21 50
tone upon steel or iron	0.30	16 40

discussion only the weight AB (Figs. 2, 3 and 4), of the body has been ed; but the body might be subjected to the action of other forces be-e force of gravity, in which case these other forces would be combined weight in order to find the resultant, this resultant being again resolved ngential and a normal component. Since the angle BAC is equal to the

angle *FEG* (Figs. 2, 3 and 4), given a certain normal pressure exerte
body on the plane, the amount of the tangential pressure T depends
angle *FEG*. The problem in actual practice reduces itself to so arran
conditions that no matter what the position of the plane may be, the
which the resultant W, makes with the normal N, to the plane, will not b
than the ANGLE OF FRICTION or REPOSE.

Theorem of the Middle Third. When any surface is subjected
sure from the action of any force or forces, this TOTAL PRESSURE may
sidered as a SYSTEM OF AN INFINITE NUMBER OF PARALLEL FORCES,
unequal in intensity. These forces will have a RESULTANT, whose MAG
DIRECTION and POINT OF APPLICATION can be determined, either graphi
by moments, as explained in Chapter VI. The determination of the
elements of this resultant force may at times become of the utmost im
to the engineer.

Pressure of this nature is technically known as the STRESS to which
face in question is subjected. (See Chapter I.) When the INTENSI
STRESS is not the same at different points of a surface, it is called a
STRESS, while if, on the contrary, its intensity remains the same at eve
of the surface, it is called a UNIFORM STRESS.

When a stress varies it may do so in one or two ways. It may v
FORMLY, that is to say, in a uniform manner, following some definit
variation, so that, knowing this law, its intensity may be determined
given point of the surface; or NON-UNIFORMLY, following no law.
stress varies in the former manner it is called a UNIFORMLY VARYING
This is the case most frequently met with in engineering problems.

Fig. 5. Resultant within Middle Third Fig. 6. Resultant at Middl

In dealing with ISOLATED FORCES, such as concentrated loads on a
are usually interested in determining the MAGNITUDE and POINT OF APP
of the RESULTANT of these forces. When, however, the question is one
or of an unlimited number of forces, the problem that usually prese
is one in which the resultant is known, in magnitude, direction and
application, and in which it is required to determine the DISTRIBUTIO
STRESS to which the surface is subjected. Or, in actual practice, it is
to so arrange the parts of the structure that this resultant shall have su
nitude, direction and point of application that the stress to which th
under consideration is subjected shall not exceed certain LIMITS or
determined beforehand by experience. For example, when the resul
known amount of pressure or stress acts at the CENTER OF GRAVITY o
face subjected to the stress, this stress is UNIFORMLY DISTRIBUTED
surface.

in the resultant acts at a distance of two-thirds the total width of the from one edge or boundary line of the surface, and at one-third the from the other edge, the stress is UNIFORMLY VARYING; and its IN- at the edge farthest from of application of the is ZERO and at the other MAXIMUM or twice the average When, however, the total of the stress remaining the the point of application of resultant is at a greater distance one edge than two-thirds the width of the surface, a certain of the surface adjacent to the farthest from the resultant is to a STRESS OF A CONTRARY to that distributed over

Fig. 7. Resultant beyond Middle Third

of the area; that is to say, if the stress to which the major part of the is subject is a COMPRESSIVE stress, the stress acting on the remainder of is a TENSILE stress. The stresses in a surface resulting from three positions of the resultant force may be illustrated graphically, as shown 5, 6 and 7. (See, also, Chapter XXXI, pages 1225 and 1234.)

2. Retaining-Walls

Definitions. A RETAINING-WALL is a wall built to resist the pressure of sand, or other filling or backing deposited behind it after it is built, as distinguished from a BREAST-WALL or FACE-WALL, which is a similar structure to prevent the fall of earth which is in its undisturbed, natural position, of which part has been excavated, leaving a vertical or inclined face. Fig. illustration of the two kinds of wall.

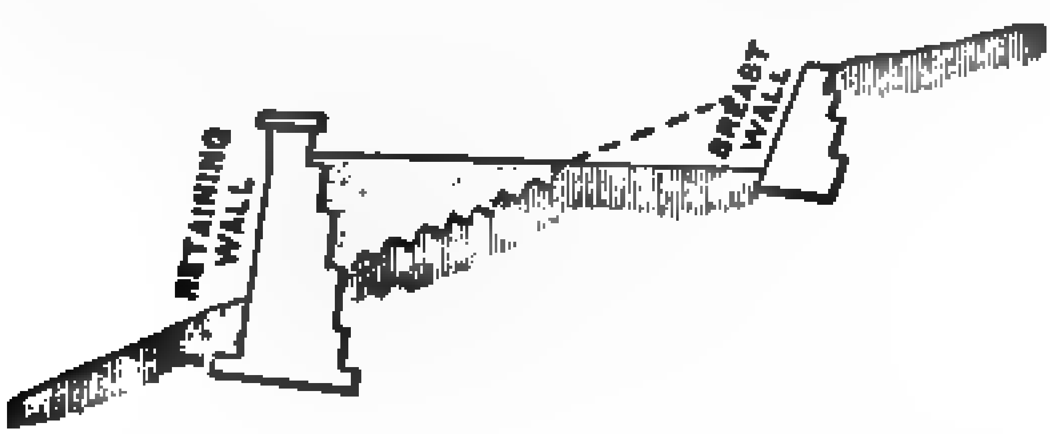

Fig. 8. Retaining-wall and Breast-wall

Theories of Retaining-Walls. A great deal has been written on the THEORY OF RETAINING-WALLS, and many theories, involving elaborate calculations for finding the CONJUGATE PRESSURES in the earth-backing behind the wall, have been developed for computing the THRUST which a bank of earth exerts against such a wall, and for determining the FORM of wall which offers the greatest resistance with the least amount of material. There are so many conditions, however, upon which the thrust exerted by the backing depends, such as the cohesion of the earth, the dryness of the material, the mode of backing up the wall, etc., that in practice it is impossible to determine the exact thrust which is exerted against a wall of a given height. It is necessary, therefore, in designing retaining-walls, to be guided by experience rather than by theory. As theories of retaining-walls are so vague and unsatisfactory, we shall not

include any in this work, but offer, rather, such suggestions, rules and
as have been established by practice and experience. A constructi
gested from empirical data, which has been found to work well in prac
determining the THRUST OF THE EARTH-BACKING and the DIMENSIONS
WALL to properly resist this thrust, is given on page 257.

In designing a retaining-wall the backing as well as the wall itself
carefully considered. THE TENDENCY OF THE BACKING TO SLIP is ver
less when the material is in a dry state than when it is saturated wit
and hence every precaution should be taken to secure good drainage.
surface-drainage, there should be openings left in the wall for the wate
may accumulate behind it to escape.

The manner in which the material is filled against the wall, also, aff
stability of the backing. If the ground is made irregular, with stepp
shown in Fig. 8, and the earth well rammed in layers inclined DOW
the wall, the pressure will be very trifling, provided that attention is
drainage. If, on the other hand, the earth is tipped in the usual ma
layers sloping DOWN TOWARDS the wall, almost the full pressure of t
will be exerted against it, and it must be made strong enough to w
such pressure.

Slopes of Repose and Angles of Repose. Cases may occur in pr
which the conditions are not such as are shown in Fig. 8, which show
limited amount of fill or new material put in behind the wall on to
original slope of the grade; cases in which, on the contrary, the wall
built on the natural surface of the ground with a view to creating an
new terrace or embankment and where all the material back of the wal

All of this material does not beat upon the wall and tend to overtu
sand or loose earth taken from an excavation and deposited on the s
the ground does not spread itself out like a liquid but piles up in a moun
PILING UP is due to the FRICTION developed between the separate pa
they slide one over the other while being dumped. This phenomenon is
in the action of any solid material broken up into separate particles; and
the SLOPE OF THE SIDES of such a mound varies with different materi
in general, the same for the same material. The angle of this slope is k
the ANGLE OF NATURAL SLOPE of the material. This angle for the mate
erally used for fill is given in the following Table II.

Table II. Slopes of Repose, Angles of Repose and Weights of L Materials

Kind of earth	Slope of repose*	Angle of repose	We lb
Sand, clean	1.5 to 1	33° 41′	
Sand and clay	1.33 to 1	36 53	
Clay, dry	1.33 to 1	36 53	
Clay, damp, plastic	2 to 1	26 34	
Gravel, clean	1.33 to 1	36 53	
Gravel and clay	1.33 to 1	36 53	
Gravel, sand and clay	1.33 to 1	36 53	
Soil	1.33 to 1	36 53	
Soft rotten rock	1.33 to 1	36 53	
Hard rotten rock	1 to 1	45 00	
Bituminous cinders	1 to 1	45 00	
Anthracite ashes	1 to 1	45 00	

* The slope is that of horizontal to vertical projection.

ssures on Retaining-Walls. Even under the conditions shown in Fig. 8, a part of the filled-in material will exert a pressure on the wall. It would atural to suppose that the part of the fill exerting pressure on the wall ill be determined by the ANGLE OF NATURAL SLOPE, all material from a al horizontal grade up to this angle being able to take care of itself, and he material above the angle needing the wall to hold it in place. Experi-t shows that this is not strictly true, for as the earth settles into place certain es of INTERNAL ELASTICITY and tendencies toward a state of EQUILIBRIUM e into play creating INTERNAL STRESSES which produce the CONJUGATE sures already referred to. The exact determination of these INTERNAL sses demands relatively complicated calculations which would be out of e in a book of this character. The construction given in the following agraphs for determining the SLOPE OF THE CLEAVAGE-PLANE, between that t of the backing which sustains itself and the triangular fill which actually s on the wall, is sufficiently accurate, however, for all practical purposes.

he Slope of the Cleavage-Plane. The following construction (Figs. 9 10), based upon empirical data, for determining first, the PRISM OF EARTH

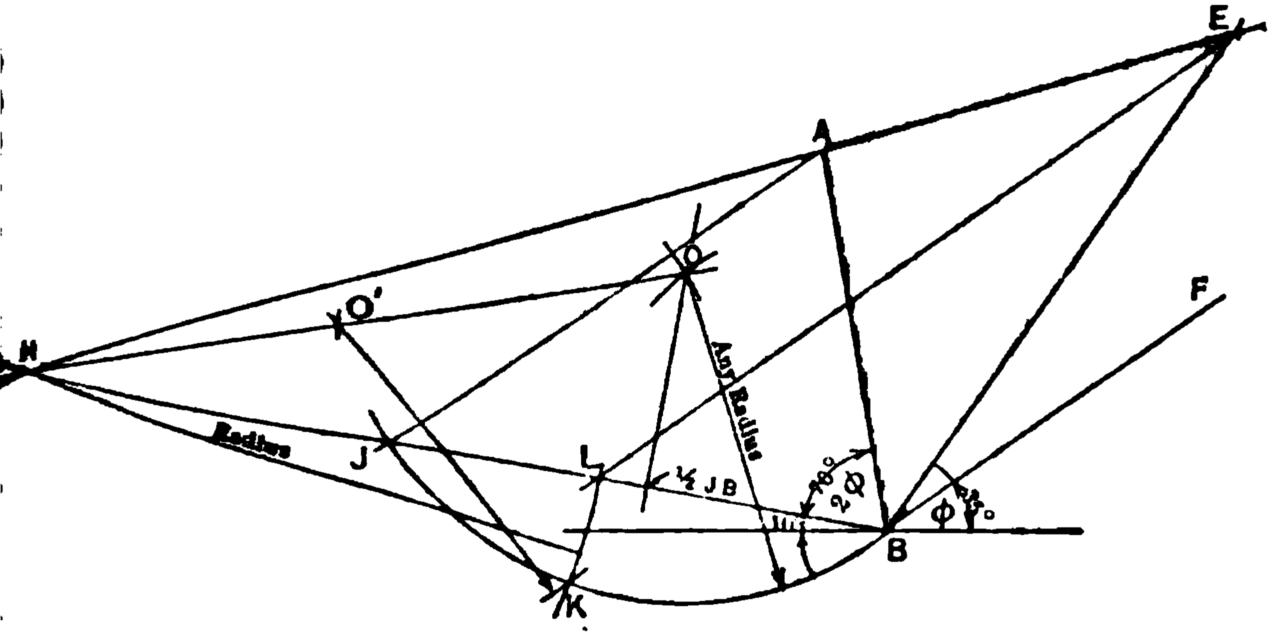

Fig. 9. Method of Determining the Prism of Earth

h exerts pressure on the back of the wall and secondly, the proper DIMEN-s for the wall, has been found to work well in practice, when certain neces-precautions are taken. These include proper DRAINAGE behind the wall, er RAMMING of the fill and efficient BRACING of the wall during its construc-.

n the calculations to determine the pressure of the earth and the weight of wall, a slice 1 ft thick is first considered. Then the area of the triangle is proportional to the volume and weight of the slice of earth causing ure on the wall, and as the area of the cross-section of the wall is propor-l to the volume and weight of the slice of the wall itself.

o determine the PRISM OF EARTH which exerts pressure against the back of wall, decide first upon the BATTER to be given to the back of the wall. In case it made 80° with the horizontal, an angle slightly greater than that ed by Trautwine. Draw *BH* (Fig. 9), making an angle *ABH*, equal to with the back of the wall; continue this line until it meets at *H* the slope e surface of the earth back of the wall, prolonged. From *A*, the top of the draw *AJ* parallel to *BF* the natural slope of the fill. This has been taken , as a fair average value. Erect a perpendicular from the middle of *JB*. With any point, *O*, as a center, on this perpendicular, describe an arc passing

through J and B. Draw HO and bisect it, and with O' as a center and O a radius, describe the arc cutting the arc JKB at K. Again, with a radius and with H as center, describe the arc KL, and finally, from L, draw LE pa to JA. The intersection of this line with the surface of the ground locate

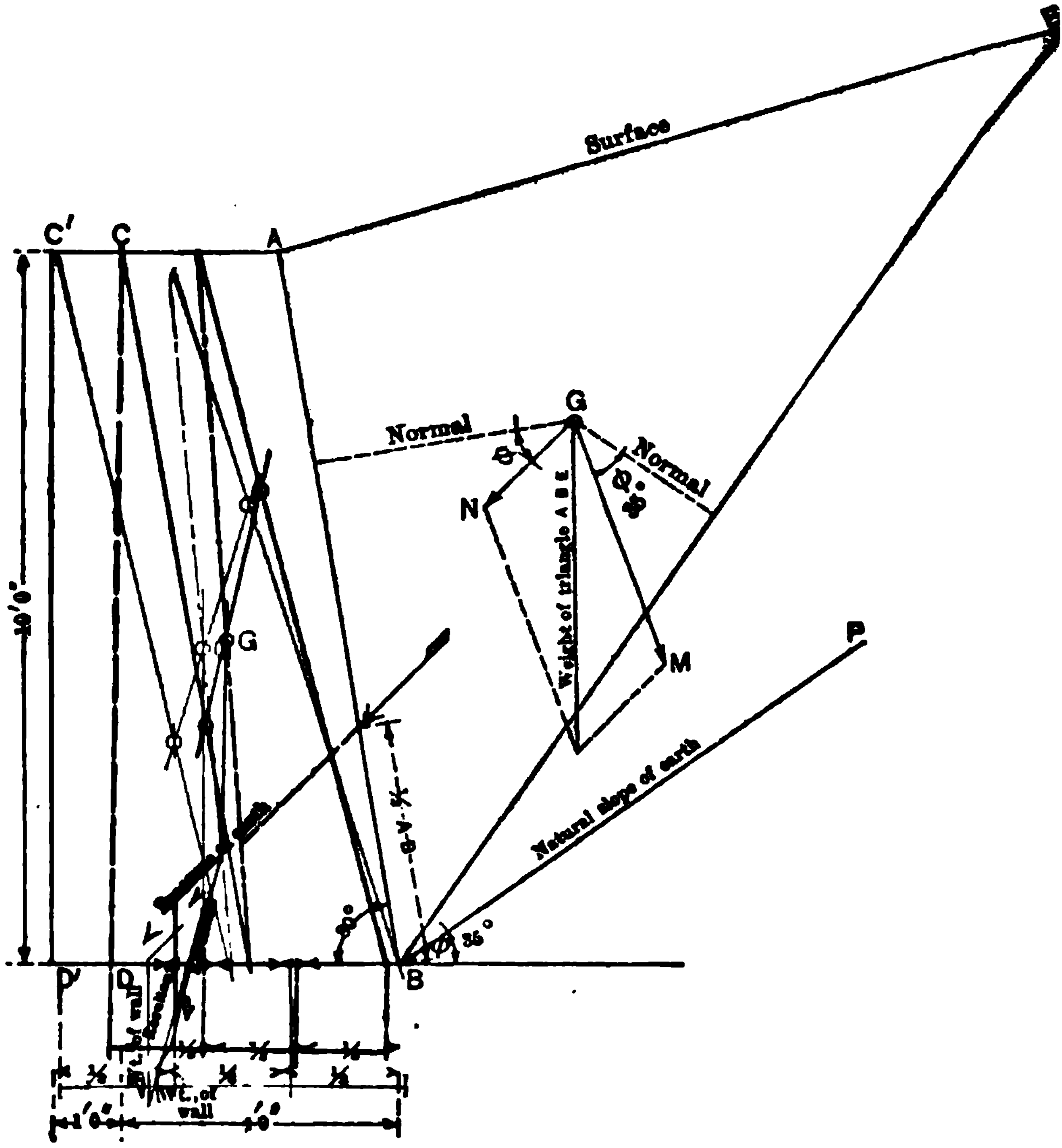

Fig. 10. Method of Determining Dimensions of Retaining-wall

point E. The line EB is the line of the CLEAVAGE-PLANE which separat part of the backing which bears against the wall from the part which exe lateral pressure.

Having found the DIMENSIONS OF THE VOLUME OF EARTH, the thrust of must be resisted by the wall, the next step is to determine what the DIMEN OF THE WALL should be to properly resist this thrust. Usually one or two are necessary before the proper solution of the problem is found. In th ample given, a preliminary trial was made with a thickness at the base o This construction is shown with the green lines (Fig. 10).

After drawing the triangle representing the base of the PRISM OF EARTH its center of gravity, G (Chap. VI). From this point draw two normals, the back of the wall and the other to the line of the CLEAVAGE-PLANE. the two lines, GM and GN, making angles ϕ with these normals. Lay o tically from the center of gravity, at any convenient scale of so many

to the linear inch, the area of the triangle of the base of the prism, the as already explained, being proportional to the volume of the prism and ight. Resolve this weight-line along the two lines GM and GN (Chap. This will give the MAGNITUDE and DIRECTION of the THRUST or pressure earth against the wall. Apply this pressure at a point on the back of the one-third of the distance from the bottom, as shown by the arrow. This the force which may tend to OVERTURN the wall and which tends to make it along the base. (See Fig. 6.)

To resist these OVERTURNING and SLIDING-TENDENCIES, the weight of the wall bined with the pressure of the earth behind it should produce a resultant satisfies the following conditions. First, its MAGNITUDE should not great enough to cause a unit pressure on the foundation-bed greater than safely bear; secondly, it should pass within the MIDDLE THIRD of the base the stress over the entire area of the base will be a COMPRESSIVE stress; thirdly, it should make an angle with a normal to the plane of the founda-bed not greater than the ANGLE OF FRICTION between the stone, brickwork, crete, or other masonry of the footings and the sand, clay, or rock of the foun-bed.

In order to determine these conditions, the CENTER OF GRAVITY of the cross-tion of the wall must be determined and a vertical line drawn through this until it intersects the line of the EARTH-THRUST produced. It is at this section of the LINES OF ACTION of the two forces that their RESULTANT To find the CENTER OF GRAVITY of the cross-section of the wall, the method dividing the trapezoid into two triangles has been followed, the center of ity of each triangle being found and these two points being joined by a line. intersection of this line with the median line drawn between the base and top of the wall is the center of gravity of the trapezoid. In this example, convenience, the scale used for the composition of the forces of the pressure the earth and the weight of the wall is one-half the scale used for the ition of the forces representing the weight of the earth-prism.

In the first trial, shown by the green lines, the first and third conditions neces-y to insure stability are fulfilled; but the second is not, the resultant pass-outside the MIDDLE THIRD of the base. This indicates, theoretically, a TENSILE stress or a tendency for the joints at the back of the wall to open. other trial, therefore, is shown with the red lines, the thickness of the wall increased as shown by the rectangle $CC'D'D$. In this second trial the OF THE WALL is necessarily increased while the EARTH-THRUST remains same. As in this case the resultant passes within the middle third, it is cluded that a wall of these dimensions, 5 ft base by 10 ft height and with batter, will be safe and will properly resist the thrust of the earth-ling.

ails of Construction. Retaining-walls are generally built with a BATTER-that is, a SLOPING face, as walls of this form are the strongest for a given of material; and if the courses are INCLINED DOWN TOWARDS THE BACK, the ncy to slide on each other will be resisted, and it will not be necessary to upon the adhesion of the mortar. The importance of making the resist-independent of the adhesion of the mortar is obviously very great, as it otherwise be necessary to delay the backing up of the wall until the mortar thoroughly set, which might require several months.

brickwork it is advisable to let every third or fourth course below the frost-project an inch or two. This increases the friction of the earth against and causes the resultant of the forces acting behind the wall to become nearly vertical, and to fall farther within the base, increasing the stability

It also conduces to strength to make the courses of varying heights throu the thickness of the wall, and to have some of the stones, especially thos the back, sufficiently high to extend through two or three courses. By means the whole masonry becomes more effectually interlocked or b together as one mass and is less liable to bulge. The courses of masonr often laid with their beds SLOPING IN, as in Fig. 15, to overcome the ten of the courses to slide on each other.

Where the ground freezes to a great depth, the back of the wall shou SLOPED FORWARD for three or four feet below its top surface, as at OC (Fig

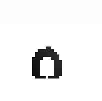

Fig. 11. Retaining-wall for Deep-freezing Earth

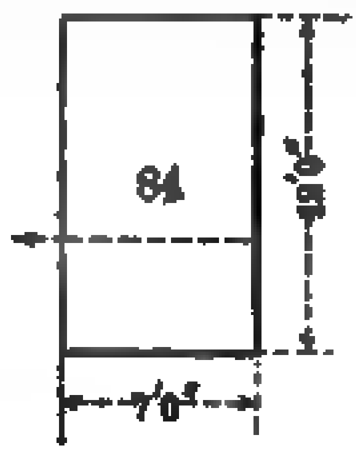

Fig. 12. Retaining-wall with Rectangular Cross-section

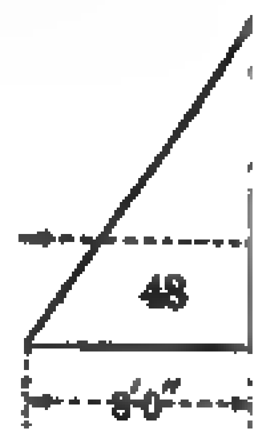

Fig. 13. Ret wall with Tri Cross-section

and this slope should be quite smooth, so as to lessen the hold of the fro prevent displacement.

Figs. 12, 13, 14 and 15 show the approximate RELATIVE VERTICAL SECT AREAS of walls of different shapes that would be required to resist the pr of a bank of earth 12 ft high. The first three examples are calculated to the maximum thrust of wet earth, while the last shows the modified form usually adopted in practice.

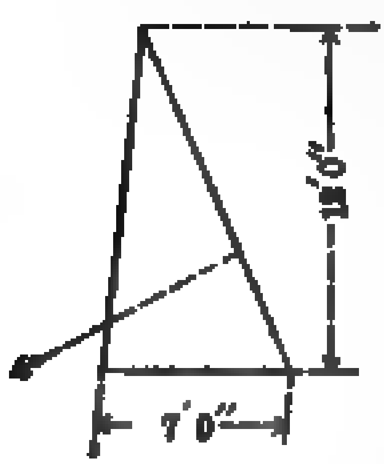

Fig. 14. Retaining-wall with Triangular Cross-section

Notes on the Thickness of Retaining-Walls. As has been stated, about the only practical rules for retaining-walls are the empirical rules based upon experience and tests. Trautwine* gives the following Table III for the thickness at the base of vertical retaining-walls with a sand backing deposited in the usual manner. The first column contains the vertical height CD (Fig. 16) of the earth as compared with the vertical height of the

Fig. 15. Ret wall with S Back

wall, AB. The latter is assumed to be 1, so that the table begins w backing of the same height as the wall. These vertical walls may be ba to any extent not exceeding 1½ in to 1 ft, or 1 in 8, without affecting stability and without increasing the base.

If the wall is built as in Fig. 17, with the ground practically level wi top, the top of the wall should be not less than 18 in thick, and the thick at a, a, etc., just above each step, should be from one-third to two-fifths

* The Civil Engineer's Pocket-Book, John C. Trautwine.

Table III. Proportions of Retaining-Walls
(Thickness of wall at the base in parts of the height, *AB*, Fig. 16)

...al height of the earth ...pared with the height ...e wall above ground	Wall of cut stone in mortar	Wall of rubble or brick, good mortar	Wall of good, dry rubble
1	0.35	0.40	0.50
1.1	0.42	0.47	0.57
1.2	0.46	0.51	0.61
1.3	0.49	0.54	0.64
1.4	0.51	0.56	0.66
1.5	0.52	0.57	0.67
1.6	0.54	0.59	0.69
1.7	0.55	0.60	0.70
1.8	0.56	0.61	0.71
2	0.58	0.63	0.73
2.5	0.60	0.65	0.75
3	0.62	0.67	0.77
4	0.63	0.68	0.78
6	0.64	0.69	0.79
14	0.65	0.70	0.80
25	0.66	0.71	0.81
or more	0.68	0.73	0.83

...s from the top of the wall to each of these levels. If the earth is banked ...n the top of the wall, the thicknesses should be increased as indicated by ...able given above. If built upon ground that is affected by frost or sur-...water, the footings should be carried sufficiently below the surface of the ...nd at the base to insure against heaving or settling.

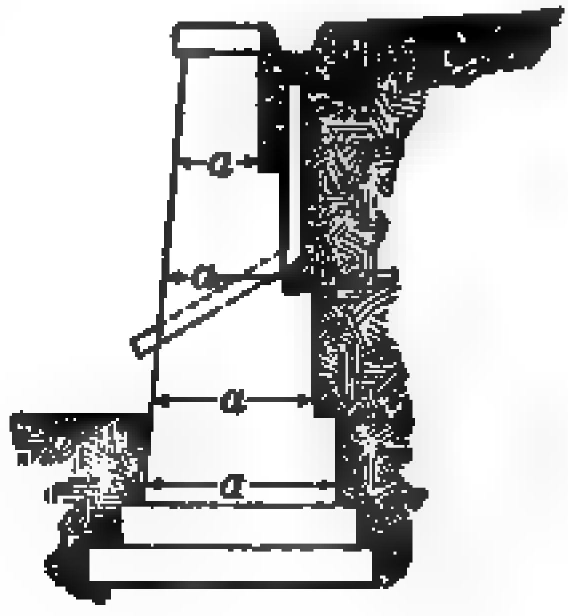

Fig. 17. Retaining-wall with Stepped Back

...s. With the constantly increasing ... purposes, there has come, also, the ...aterial. Figs. 18,* 19* and 20* ... for retaining-walls to satisfy the

..., Taylor and Thompson.

requirements of banks 5, 10 and 20 ft high. The wall shown in Fig
reinforced at intervals with COUNTERFORTS. The walls themselves in F
and 19 act as CANTILEVER BEAMS The FOOTINGS, in all three cas
subjected to two principal external forces, the resultant of the u

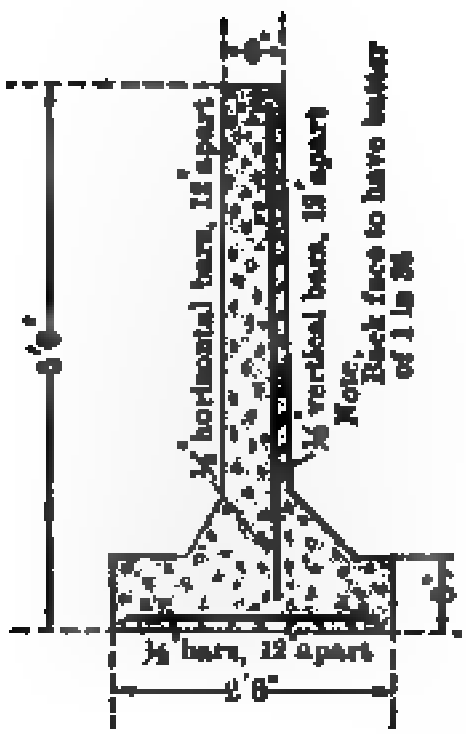

Fig. 18. Reinforced-concrete Retaining-wall, 5 ft High

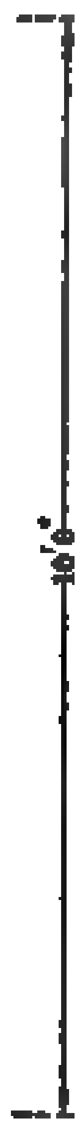

Fig. 19. Reinforced-concrete Retaining-w ft High

upward pressure of the foundation-bed and the resultant of the
ward pressures of the fill. In Fig. 20 the COPING acts as a BEAM F
BOTH ENDS, with a span equal to the distance between the count
and loaded with the proper proportion of the load due to the pres
the fill behind the wall and transmitted to the coping by the wall. T
itself in this case acts as a FLOOR-SLAB supported on all four sides and su
to an approximately evenly distributed load. The counterforts are in
The MAXIMUM BENDING MOMENTS for these various cases can be det
(Chapter IX) and the necessary DIMENSIONS and REINFORCEMENT to
vided decided by the rules given in Chapter XXIV.

3. Breast-Walls

Breast-Walls. Where the ground to be supported is firm, and the st
horizontal, the office of a BREAST-WALL (Fig 8) is more to protect than to
the earth. It should be borne in mind that a trifling force skilfully applie
broken ground will keep in its place a mass of material, which, if once al
move, would crush a heavy wall. Great care, therefore, should be take

the newly opened ground to the influence of air and water longer than is for sound work, and to avoid leaving the smallest space for motion the back of the wall and the ground. The strength of a breast-wall must rtionately increased when the strata to be supported incline down

Fig. 20. Reinforced-concrete Retaining-wall with Counterforts and Apron

the wall; where they incline down from it, the wall need be little more thin facing to protect the ground from disintegration. The preserva- f the natural drainage is one of the most important points to be d to in the erection of breast-walls, as upon this their stability in a easure depends. No rule can be given for the best way to do this; it tter for attentive consideration in each particular case.

4. Vault-Walls

-Walls. In large cities it is customary to utilize the space under the t for storage or other purposes. This necessitates a wall at the curb- hold back the earth and the street-pressures and also the weight of the . Where practicable the space should be divided by partition-walls y 10 ft, and when this is done the outer wall may be advantageously —d bricks in the form of arches, as shown in Fig. 21. The thickness

of the arch should be at least 16 in for a depth of 9 ft and the rise (arch from one-eighth to one-sixth of the span. If partitions are not p cable, each sidewalk-beam be supported by a heavy I column, with either flat o mental arches between, of brick or concrete. Fig. shows a detail of the outer of the vault under the sic around the Singer building York City. These walls (of a core formed by tw brick arches with vertical built between the flang 8-in vertical steel I spaced about 5 ft apa bedded at the bottom in crete footing. Their to joined by 6-in horizontal I beams and braced laterally by the sidewalk- 5 ft apart. The arches themselves are segmental, with a rise of abou

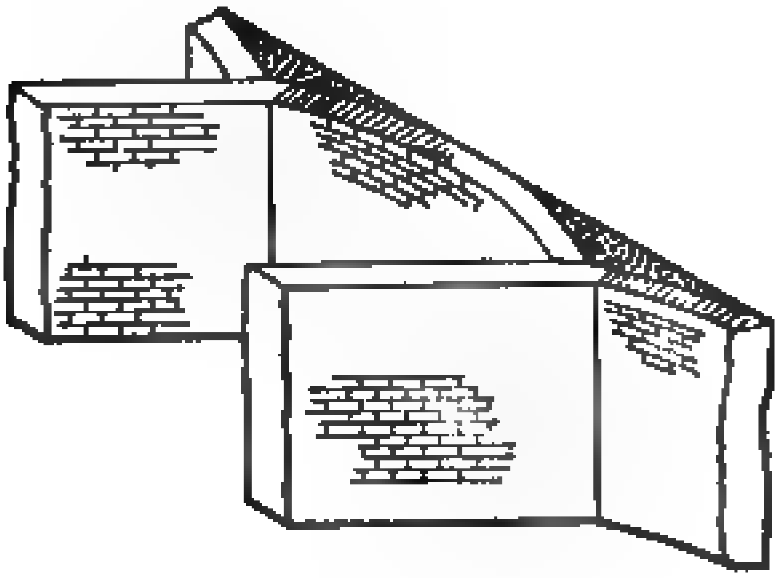

Fig. 21. Vault-wall with Partitions

Fig. 22. Vault-walls of Singer Building, New York City

and are built up solid against an 8-in outside face-wall. A 4-in plain wall is built inside against the flanges of the vertical beams, inclos mental air-chambers in front of each arch.

* From The Engineering Record, Feb. 26, 1898.

CHAPTER V

[ST]RENGTH OF BRICKS, STONE, MASS-CONCRETE AND MASONRY

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1. Crushing Strength of Stonework, Brickwork, Bricks, etc.

[Stre]sses in Masonry. By the term STRENGTH OF MASONRY is generally [mean]t its resistance to a direct COMPRESSIVE force or load, and this is the only [kind of] stress to which masonry should be subjected. Stone lintels and footings [may] be subjected to a TRANSVERSE or BENDING STRESS, but they can hardly be [inclu]ded in the term masonry, as they consist of single pieces. There are also [tenden]cies to bend and to split apart in brick walls and piers, as they are usually [h]i[gh] in proportion to their lateral dimensions, but the stresses thus developed [canno]t be accurately determined and should be avoided as much as possible. [It is] impossible to fix values for the strength of brickwork or stonework with [anyth]ing like the exactness possible for wooden or steel members, for the reason [that] there is not only a great variation in the strength of different kinds of [brick] and stone, even when taken from the same kiln or quarry, but the strength [of wa]lls and piers is also greatly affected by the kind and quality of the mortar [used], the way in which the work is built and bonded, and the amount of moisture [in th]e materials when they are laid. All that can be done, therefore, is to give [value]s which will be safe for the different kinds of masonry built in the usual [mann]er.

[Wor]king Compressive Strength of Masonry. The building laws of most [of th]e larger cities of this country specify the maximum loads per square foot [allow]ed to be placed upon different kinds of masonry, and these laws must govern [arch]itects in such cities. When there is no restriction of this kind, Table I [give]s a pretty good idea of the maximum loads which it is safe to put upon the [differ]ent kinds of work mentioned. Table II gives the maximum safe loads [specif]ied in the building laws of several cities, and the remaining tables of the [chapt]er give records of numerous tests made to determine the ultimate com[press]ive strengths of various kinds of bricks, building stones, mortars and con[crete]s, and are of value in determining the safe loads for special cases. In [deter]mining the safe compressive resistance of masonry from tests on the ulti[mate] compressive strength of work of the same kind, a factor of safety of at [least] 10 should be allowed for piers and 20 for arches.

Table I. Safe Working Loads for Masonry

BRICKWORK IN WALLS OR PIERS

	Tons per square foot Eastern	Tons per square foot Western
[Common b]ricks in lime mortar	7	5
[Common b]ricks in hydraulic-lime mortar	...	6
[Common b]ricks in natural-cement mortar, 1 : 3	10	8
[Hard o]r pressed bricks in lime mortar	8	6
[Hard o]r pressed bricks in natural-cement mortar	12	9
[Hard o]r pressed bricks in Portland-cement mortar	15	12½

Piers exceeding in height six times their least lateral dimensions sho increased 4 in in lateral dimensions for each additional 6 ft.

Stonework

	To squa
Rubble walls, irregular stones	
Rubble walls, coursed, soft stone	
Rubble walls, coursed, hard stone	5 to
Dimension-stone, squared, in cement mortar:*	
Sandstone and limestone	10 to
Granite	20 to
Dressed stone, with ⅜-in dressed joints, in Portland-cement mortar:*	
Granite	
Marble or limestone, best	
Sandstone	

The height of columns should not exceed eight times the least diameter, the least diameter is sufficiently greater than necessary for the strength material used.

Concrete †

Portland-cement mortar, 1 : 8, 6 months, 10 tons; 1 year, 15 to 20 t
Natural-cement mortar, 1 : 6, 6 months, 3 tons; 1 year, 5 to 8 tons

Hollow tile

Safe loads per square inch of effective bearing parts

Hard fire-clay tiles	80 l
Hard ordinary clay tiles	60 l
Porous terra-cotta tiles	40

Mortar

In ½-in joints, 3 months old

	To squ
Portland-cement mortar, 1 : 4	
Natural-cement mortar, 1 : 3	
Lime mortar, best	8 to
Portland-cement mortar, 1:2, ¼-in joints, bedding iron plates	

The values given above are generally very conservative. The leading tects and engineers of Chicago recommended for that city in 1908 the ing SAFE WORKING PRESSURES for brick and stone masonry and concrete:

Common bricks, crushing strength 1 800 lb per sq in:	Lb per sq in	T
In lime mortar	100	
In lime-and-cement mortar	125	
In natural-cement mortar	150	
In Portland-cement mortar	175	

* Limestone ashlar is usually set in (1) lime mortar, (2) puzzolan, natural mortar, or other non-staining cement mortar, or (3) mortar composed of lime ar Portland cement.

† See pages 283 to 287.

	Lb per sq in	Tons per sq ft
...ct, hard, common bricks, crushing strength equal to 2 500 lb per sq in:		
...n 1 part Portland cement, 1 lime-paste and 3 sand..	175	12⅗
...n 1 : 3 Portland-cement mortar..................	200	14⅖
...ed and sewer-bricks, crushing strength equal to 5 000 lb per sq in: 1 : 3 Portland-cement mortar..	250	18
...ing bricks, in 1 : 3 Portland-cement mortar.......	350	25⅕
...crete, natural cement, 1 : 2 : 5....................	150	10⅘
...crete, Portland cement, 1 : 3 : 6, machine-mixed..	300	21⅗
...crete, Portland cement, 1 : 3 : 6, hand-mixed.....	250	18
...crete, Portland cement, 1 : 2 : 4, machine-mixed...	400	28⅘
...crete, Portland cement, 1 : 2 : 4, hand-mixed......	350	25⅕
...ble, uncoursed, in lime mortar..................	60	4⅓
...ble, uncoursed, in Portland-cement mortar.......	100	7⅕
...ble, coursed, in lime mortar....................	120	8⅗
...ble, coursed in Portland-cement mortar..........	200	14⅖
...lar, limestone, in Portland-cement mortar........	400	28⅘
(See note on limestone ashlar, page 266.)		
...lar, granite, in Portland-cement mortar..........	600	43⅕
...mittee on Code Requirements for Indiana limestone recommends:		
...estone-ashlar masonry, in lime mortar, equivalent to 1 : 3 cement-and-lime mortar...........	200 to 250	14⅖ to 18
...natural-cement-and-lime mortar, 1 : 3............	400 to 500	28⅘ to 36

Table II. Comparison of Building Laws *

Materials	Boston, 1915	Buffalo, 1909	New York, 1917	Chicago, 1916	St. Louis, 1907	Philadelphia, 1914	Denver, 1898
	Allowable pressures in tons per sq ft						
...ite, cut...............	60–72	.....	72	43	.....	.....	40
...ble and limestone, cut...	40	.....	43–50	29	.....	.....	.....
...stone, hard cut.........	30	.....	29	29	.....	.....	12
...-burned bricks in Portland-cement mortar.......	18–20	12	18	12½	21½	.....	.....
...-burned bricks in natural-cement mortar.......	.....	9	15	10½	.....	15	9
...rd-burned bricks in cement-and-lime mortar...	12–14	.....	11½	.....	11½	12	.....
...-burned bricks in lime mortar.................	6–8	6	8	7	11	8	8
...ed bricks in Portland-cement mortar...........	.....	12	.....	18	.....	.....	.....
...d bricks in natural-cement mortar...........	.....	9	.....	.....	.....	.....	12
...le stone in natural-cement mortar...........	.....	5†	8‡	.....	.....	10	12
...nd-cement concrete in foundations, 1 : 2 : 4.....	25–30	4	36	18–28§	18	15	10
...al-cement concrete in foundations, 1 : 2 : 4.....	.....	.....	15	10½‖	.....	.....	4

...notes, page 287, relating to building laws and working loads for masonry, etc.
...Portland-cement mortar. ‡ In Portland-cement mortar, 10; in lime-cement ...7.
...ding to mixture. ‖ 1 : 2 : 5 mixture.

Brick Piers. As a rule brickwork is subject to its full safe resistanc when used in piers, and in small sections of walls, under bearing-plates. latter case but a few courses receive the full load, and hence a greate stress may be allowed than for piers. Values for computing the area of be plates are given in Chapter XIII. Aside from the quality of the wor materials the two elements which most influence the strength of brick are the ratio of height to least lateral dimension and the method of be When the height of a brick pier exceeds six times its least lateral dimensi load per square foot should be reduced from the values given in Table I.

Formulas for the Safe Strength of Brick Piers exceeding six dia in height. From the records of numerous tests on the strength of brick from some formulas published * by Ira O. Baker, and also from personal vation, Mr. Kidder deduced the following formulas for the maximum w loads for first-class brickwork in piers whose height exceeds six times th lateral dimension.

For piers laid with rich lime mortar:

$$\text{Safe load per square inch} = 110 - 5\ H/D$$

For piers laid with 1 : 2 natural-cement mortar:

$$\text{Safe load per square inch} = 140 - 5\tfrac{1}{2} H/D$$

For piers laid with 1 : 3 Portland-cement mortar:

$$\text{Safe load per square inch} = 200 - 6\ H/D$$

H representing the height in feet, and D the least lateral dimension in fe

For a pier 20 ft high and 2 ft square these formulas will reduce the s to 4.3 tons per sq ft for lime mortar, 6.1 tons for natural-cement mor 10 tons for Portland-cement mortar. No pier over 8 ft high should be l 12 by 12 in in cross-section and when from 6 to 8 ft high piers should be 8 by 12 in in cross-section.

The following is the Chicago law (1914): "Isolated piers of concret or masonry shall not be higher than six times their smallest dimension the above unit stresses ‡ are reduced according to the following formula

$$P = C\,(1.25 - H/20\,D)$$

in which P is the reduced allowed unit load, C the unit stress above to, H the height of the pier in feet and D the least dimension of the pie No pier shall exceed in height twelve times the least dimension. Th of the pier shall be added to other loads in computing the load on the

Brick piers intended to carry more than 50% of the safe loads give should not be built in freezing weather nor with dry bricks. Lime should not be used for building piers that are to receive their full loa three months.

Effect of Bond on the Strength of Brickwork. Brick piers, l the point of destruction, always fail by the splitting and bulging ou

* In the Brickbuilder, April, 1898.

† For piers faced with pressed bricks, laid with joints ¼ in or less in thick backed with common bricks in lime mortar, only the dimensions of the backing considered in figuring their strength. If the backing is laid in cement morta face-bricks are well tied to the backing, the full section of the pier may be consid piers veneered with stone or terra-cotta, 4 in thick, only the strength of the back be considered.

‡ These are in general the "safe working pressures" for brickwork previously as recommended by the Chicago architects and engineers in 1908.

...selves, and not by direct crushing of the bricks or mortar, showing ...s are weakest in their bond and in the tensile or transverse strengths of ...ks. It is very important, therefore, to have the brickwork well bonded, ...l joints filled with mortar or grouted. The strength of a brick pier in... to carry an extreme load would probably be increased by bonding fre... with hoop-iron in addition to the regular brick-bond.*

...d-Stones in Brick Piers. Many competent architects and builders ...r that the strength of a brick pier is increased by inserting bond-stones, ...5 to 8 in in thickness and the full size of the pier in cross-section, every 3 or ... height.

...example, the Building Laws for the City of New York (1916) require ...stones every 30 in in height, and at least 4 in in thickness, to be built into ...piers which contain less than 9 superficial feet of section, and which sup... beam, girder, arch, or column on which a wall rests, or lintel spanning ...ning over 10 ft and supporting a wall. The New York laws allow per...d steel or cast-iron plates of the full cross-section of the pier to be used ...d of the bond-stones. On the other hand, there are many first-class ...s who consider that bond-stones in a brick pier do more harm than good, ...he author is of the opinion that this is generally the case. The Boston ...ng Laws do not require intermediate bond-stones. If bond-stones are ..., they should be bedded so as to bear rather more heavily on the inner ...n of the pier than on the outer 4 in, for unless this is done the outer shell ...the most of the load, and will be likely to bulge away from the core. A ...hich supports a girder or column should have a cap-stone or iron plate of ...ent strength to distribute the pressure over the cross-section of the pier.

...lls Faced with Stone, Terra-Cotta, or Cement Blocks. Brick walls ... with blocks or ashlar of any material should always have the backing laid ...ment mortar or in cement-and-lime mortar, unless the backing is very ..., that is, 30 in or more. The aggregate thickness of the mortar joints in ...acking is so much greater than in the facing, that any shrinkage or com...ion of the mortar tends to throw undue weight on the facing and to sepa...t from the backing. Veneering generally should be tied to the backing ...t every 18 in in height. Stone courses, up to about 36 in in height, ... have anchors in the bed-joints only. Anchors are placed in the side ... also, when the height of the courses is more than 36 in. The Building ...ances of several large cities require that all bearing walls faced with ... laid in running bond, and all walls faced with stone ashlar less than 8 in ... shall be of such thickness as to make the wall independent of the facing ...n to that required for unfaced walls. Ashlar 8 in thick and bonded into ...cking may be counted as part of the thickness of the wall.

...uting.† It is contended by persons having large experience in building ...asonry carefully grouted, when the temperature is not lower than 40° F., ...e the most efficient result. Many of the largest buildings in New York ...ave grouted walls. The Mersey docks and warehouses at Liverpool, ...d, one of the greatest pieces of masonry in the world, were grouted ...hout. There are many engineers and others who do not believe in ...g, claiming that the materials tend to separate and form layers.

...hing Height of Bricks and Stone. If we assume that the weight of ...ork is 120 lb per cu ft, and that it would commence to crush under 700 lb

* ...e manner in which brick piers fail is excellently shown by illustrations on page 79 ...rickbuilder for May, 1896.

† ...e American Architect, July 21, 1887, page 11.

per sq in, then a wall of uniform thickness would have to be 840 ft high the bottom courses would commence to crush from the weight of the bric above. Average sandstones, at 145 lb per cu ft, would require a column 5 high to crush the bottom stones, and an average granite, at 165 lb per would require a column 10 470 ft high. The Merchants' shot-tower at more is 246 ft high, and its base sustains a pressure of 6½ tons per sq ft, tl being long tons of 2 240 lb. The base of the granite pier of Saltash Brid Brunel), of solid masonry to the height of 96 ft, and supporting the ends iron spans of 455 ft each, sustains 9½ tons per sq ft.

Stone Piers. Piers of good strong building stone laid in courses t cross-sections of the piers, with the top and bottom courses bedded tr even, may be built to support very heavy loads. The height of such however, should not exceed ten times the least lateral dimension, and w exceeds eight times the thickness, the load should be reduced. The should not exceed ⅜ in in thickness and should be spread with 1 : 2 Po cement mortar, kept back 1 in from the face of the pier to prevent spal the edges. A test of the strength of a limestone pier 12 in square is de under Marbles and Limestones, in this chapter. Rubble-work should used for piers whose height exceeds five times the least dimension, or in the latter is less than 20 in.

Records of Tests on the Crushing Resistance of Bricks. Tal gives the results of some tests on bricks, made under the direction of Mr. in behalf of the Massachusetts Charitable Mechanics' Association.

Table III. Ultimate and Cracking Strengths of Bricks

Kind of brick	Size of test-specimen	Area of face, sq in	Commenced to crack under lb per sq in
Philadelphia face-bricks..........	Whole bricks	33.7	4 303
Philadelphia face-bricks..........	Whole bricks	32.2	3 400
Philadelphia face-bricks..........	Whole bricks	34.03	2 879
Average....................	....	....	3 527
Cambridge bricks (Eastern)......	Half-bricks	10.89	3 670
Cambridge bricks (Eastern)......	Whole bricks	25.77	7 760
Cambridge bricks (Eastern)......	Half-bricks	12.67	3 393
Cambridge bricks (Eastern)......	Half-bricks	13.43	3 797
Average....................	....	....	4 655
Boston Terra-Cotta Co.'s bricks..	Half-bricks	11.46	11 518
Boston Terra-Cotta Co.'s bricks..	Whole bricks	25.60	8 593
Boston Terra-Cotta Co.'s bricks..	Whole bricks	28.88	3 530
Average....................	....	....	7 880
New England pressed bricks.....	Half-bricks	12.95	3 862
New England pressed bricks.....	Half-bricks	13.2	8 180
New England pressed bricks.....	Half-bricks	13.30	2 480
New England pressed bricks.....	Half-bricks	13.45	4 535
Average....................	....	....	4 764

specimens were tested in the government testing-machine at Watertown, and great care was exercised to make the tests as perfect as possible. parallel plates between which the bricks are crushed are fixed in one position it is necessary that each specimen tested should have perfectly parallel The bricks which were tested were rubbed on a revolving bed until the bottom faces were perfectly true and parallel. The preparation of the in this way required a great deal of time and expense; and it was so difficult some of the harder bricks that they had to be broken and only one-half brick prepared at a time.

Philadelphia bricks used in these tests were obtained from a Boston and were fair samples of what is known in Boston as Philadelphia Face- They were very soft bricks.

Cambridge bricks were the common bricks, such as are made around They are about the same as the Eastern bricks.

Boston Terra-Cotta Company's bricks were manufactured of a rather and were such as are often used for face-bricks.

New England pressed bricks were hydraulic-pressed bricks, and were as hard as iron.

tests made on the same machine by the United States Government in the average strength of three (M. W. Sands) Cambridge, Mass., face- was 13 925 lb, and of his common bricks, 18 337 lb per sq in, one brick the enormous strength of 22 351 lb per sq in. This was a very hard- brick. Three bricks of the Bay State (Mass.) manufacture showed an strength of 11 400 lb per sq in. The New England bricks are among and strongest in the country, those in many parts of the West not one-fourth the strength given above; so that in heavy buildings, where strength of the bricks to be used is not known by actual tests, it is advisable the bricks tested. Ira O. Baker reported some tests on Illinois bricks, on the 100 000-pound testing-machine at the University of Illinois in 1889, which give for the crushing strength of soft bricks, 674 lb per sq in, average of three face-bricks, 3 070 lb per sq in, and for four paving- 9 775 lb per sq in. In nearly all makes of bricks it will be found that the bricks are not as strong as the common bricks.

of the Strength of Brick Piers Laid with Various Mortars.* tests were made for the purpose of testing the strength of brick piers with different cement mortars, as compared with those laid up with mortar. The bricks used in the piers were procured at M. W. Sands's Cambridge, Mass., and were good ordinary bricks. They were from lot as the samples of common bricks described above. The piers by 12 in in cross-section, and nine courses, or about 22½ in high, except- first, which was but eight courses high. They were built Nov. 29, 1881, of the storehouses at the United States Arsenal in Watertown, Mass. In have the two ends of the piers perfectly parallel surfaces, a coat of pure cement, about ½ in thick, was put on the top of each pier and the foot in the same cement. On March 3, 1882, three months and five the tops of the piers were dressed to plane surfaces at right-angles sides of the piers. On attempting to dress the lower ends of the piers, grout peeled off, and it was necessary to remove it entirely and put of cement similar to that on the tops of the piers. This was allowed for one month and sixteen days, when the piers were tested. At that piers were four months and twenty-six days old. As the piers were cold weather, the bricks were not wet. They were built by a skilled

* Made under the direction of F. E. Kidder.

bricklayer and the mortars were mixed under his superintendence. Th
were made with the government testing-machine at the Arsenal. The
ing table is arranged so as to show the result of these tests, and to afford
means of comparison of the strength of brickwork with different mortars
piers generally failed by cracking longitudinally, and some of the bricl
crushed. The Portland cement used in these tests was made by Brooks
bridge & Company, of England. Roman cement is a European natural
usually, although not always, containing a low percentage of magne
sets rapidly, has about one-third the strength of true Portland cement
much weakened by the addition of sand.

Table IV. Tests of Piers of Common Bricks Laid in Different Mo

Piers 8 by 12 in in section, built of common bricks.	Ultimate strength of pier, lb	Pressure per sq in under which pier commenced to crack, lb	Ult str lb
Lime mortar	150 000	833	
Lime mortar, 3 parts; Portland cement, 1 part	290 000	1 875	
Lime mortar, 3 parts; Newark and Rosendale cements, 1 part	245 000	1 354	
Lime mortar, 3 parts; Roman cement, 1 part	195 000	1 041	
Portland cement, 1 part; sand, 2 parts	240 000	1 302	
Newark and Rosendale cements, 1 part; sand, 2 parts	205 000	708	
Roman cement, 1 part; sand, 2 parts	185 000	1 770	

As the actual strength of brick piers is a very important consider
building-construction, some tests, made by the United States Govern
Watertown, Mass., and contained in the report of the tests made on the
ment testing-machine for the year 1884, are given as being of muc
Three kinds of bricks were represented in the construction of the pi
mortars of different composition, ranging in strength from lime morta
Portland-cement mortar. The piers ranged in cross-section dimensi
8 by 8 to 16 by 16 in, and in height from 16 in to 10 ft. They were
the age of from 18 to 24 months.

Table V gives the results obtained and memoranda regarding the
character of the piers.

Table VI gives the results obtained from tests of the strength of br
made at the McGill University, Montreal, laboratories, in March, 1897

Recent Tests of Brick Piers.* Elaborate tests of brick piers, with
results,† were made in 1908 by A. N. Talbot and D. A. Abrams at
versity of Illinois Experiment Station. Table VII is a summary of thos
The tests were made on sixteen brick piers, the lengths of which va

* See, also, results of important tests made in 1914 and 1915 at Columbia U
New York, by J. S. Macgregor.

† Bulletin 27, University of Illinois Engineering Experiment Station, Sept. 2

Number of test	Nominal dimensions: Height ft	Nominal dimensions: Height in	Nominal dimensions: Cross-section	Composition of mortar	Weight per cubic foot lb	Sectional area sq in	First crack lb	Ultimate strength: Total lb	Ultimate strength: Lb per sq in	Ultimate strength: Tons per sq ft	Ultimate strength: Per cent of single brick
Built of face-bricks (M. W. Sands, Cambridge, Mass.)											
11	1	4	8 by 8	1 lime mortar, 3 sand	137.4	57.00	85 000	143 600	2 520	181.4	18.1
320	6	8	8 by 8	1 lime mortar, 3 sand	133.5	57.76	50 000	108 400	1 877	135.1	13.5
12	1	4	8 by 8	1 Portland-cement mortar, 2 sand	136.3	57.76	200 000	218 100	3 776	271.8	27.1
321	6	8	8 by 8	1 Portland-cement mortar, 2 sand	133.5	57.76	85 000	129 900	2 249	161.9	16.2
283	2	0	12 by 12	1 lime mortar, 3 sand	....	132.25	140 000	257 100	1 940	139.7	13.9
284*	2	0	12 by 12	1 lime mortar, 3 sand	....	113.76	90 000	226 100	1 990	143.3	14.3
332	10	0	12 by 12	1 lime mortar, 3 sand	131.7	132.25	70 000	199 800	1 511	108.8	10.9
334†	10	0	12 by 12	1 lime mortar, 3 sand	125.0	115.44	100 000	208 600	1 807	130.1	13.0
286	2	0	12 by 12	1 Portland-cement mortar, 2 sand	....	132.25	200 000	486 000	3 670	264.2	26.4
326	10	0	12 by 12	1 Portland-cement mortar, 2 sand	132.2	132.25	200 000	298 000	2 253	162.2	16.2
Built of common bricks (M. W. Sands)											
10	1	4	8 by 8	1 lime mortar, 3 sand	135.6	60.80	66 000	148 800	2 440	175.6	13.3
12½	6	8	8 by 8	1 lime mortar, 3 sand	133.6	62.40	....	96 100	1 540	110.8	8.4
281	2	0	12 by 12	1 lime mortar, 3 sand	....	138.06	75 000	296 400	2 150	154.8	11.7
282‡	2	0	12 by 12	1 lime mortar, 3 sand	....	119.58	120 000	244 600	2 050	147.6	11.2
331	9	9	12 by 12	1 lime mortar, 3 sand	131.5	138.06	70 000	154 300	1 118	80.5	6.1
330§	10	0	12 by 12	1 lime mortar, 3 sand	136.0	115.50	70 000	183 300	1 587	114.3	8.6
329	10	0	12 by 12	1 Portland-cement mortar, 2 sand	131.0	138.06	....	276 600	2 003	144.2	10.9
387	2	8	16 by 16	1 Portland-cement mortar, 2 sand	....	256.00	460 000	696 000	2 720	195.8	14.8
328	10	0	16 by 16	1 Portland-cement mortar, 2 sand	....	256.00	340 000	483 100	1 887	135.8	10.3

Number of test	Nominal dimensions			Composition of mortar	Weight per cubic foot	Sectional area	First crack	Ultimate strength			
	Height		Cross-section					Total	Lb per sq in	Tons per sq ft	Per cent of single brick
	ft	in			lb	sq in	lb	lb			
				Built of common bricks (Bay State)							
285	2	0	12 by 12	1 lime mortar, 3 sand	....	146 41	95 000	201 000	1 370	98 6	12.0
288	6	0	12 by 12	1 lime mortar, 3 sand	....	144 00	70 000	163 200	1 133	81 6	9 9
289	6	0	12 by 12	1 lime mortar, 3 sand	119 7	144 00	100 000	174 300	1 210	87 1	10.6
291*	6	0	12 by 12	1 lime mortar, 3 sand	118.2	144 00	80 000	191 600	1 331	95 8	11 7
292†	6	0	12 by 12	1 lime mortar, 3 sand	118 1	156 25	110 000	189 200	1 211	87.2	10.6
325	7	10	12 by 12	1 lime mortar, 3 sand	120.0	144 00	100 000	169 100	1 174	84.6	10.3
327	10	0	12 by 12	1 lime mortar, 3 sand	118.0	144 00	90 000	133 100	924	66 6	8.1
335	10	0	8 by 12	1 lime mortar, 3 sand	107.0	96 00	35 000	90 200	940	67.7	8.2
333	10	0	12 by 16	1 lime mortar, 3 sand	118.7	192 00	80 000	148 500	773	55 7	6.8
301	6	0	12 by 12	1 Rosendale-cement mortar, 2 lime mortar	120.6	144 00	160 000	237 000	1 646	118 5	14.4
293	6	0	12 by 12	1 Rosendale-cement mortar, 2 sand	123 0	144 00	260 000	284 000	1 972	142.0	17.3
300	6	0	12 by 12	1 Portland-cement mortar, 2 lime mortar	120.3	144 00	150 000	203 200	1 411	101.6	12 4
294	6	0	12 by 12	1 Portland-cement mortar, 2 sand	119 7	144 00	220 000	258 000	1 792	129 0	15 7
290	6	0	12 by 12	Neat Portland-cement mortar	126 6	144 00	280 000	342 000	2 375	171.0	20.8

* Joints broken every six courses. † Bricks laid on edge.

[...] VI. Tests of Brick Piers, McGill University Laboratories, March, 1897

[...]ensions of piers	Composition of mortar	Kind of bricks	Crushing strength, lb per sq in		Age
			At first crack	Maximum load	
[...] by 8.1 in, 11.6 [...] high; joints [...] in thick	1 Canadian Portland-cement mortar, 3 sand	Ordinary well-burned flat bricks	822	1 234	3 weeks
[...] by 8.1 in, 11.6 [...] high; joints [...] in thick	1 German Portland-cement mortar, 3 sand	Ordinary well-burned flat bricks	990	1 230	3 weeks
[...] by 8.3 in, 10.5 [...] high; joints [...] in thick	1 English Portland-cement mortar, 3 sand	La Prairie pressed bricks, keyed on one side	1 130	1 524	3 weeks
[...] by 8.4 in, 10.75 [...] high; joints [...] in thick	1 Belgian Portland-cement mortar, 3 sand	La Prairie pressed bricks, keyed on one side	1 204	1 985	3 weeks

[...]able VII. Tests of Brick Piers, Made at the University of Illinois

The amounts given are average values

[...]aracteristics of piers	Average unit load lb per sq in	Ratio of strength of pier to strength of brick	Ratio of strength of pier to strength of first of series	Crushing strength of 6-in mortar-cubes lb per sq in	Ratio of strength of pier to strength of cubes
Shale building bricks					
[...] laid, 1 : 3 Portland-[...]ent mortar, 67 days	3 363	0.31	Standard 1.00	2 870*	1.17
[...] laid, 1 : 3 Portland-[...]ent mortar, 6 months.	3 950	0.37	1.18	....	....
[...] laid, 1 : 3 Portland-[...]ent mortar, eccentri-[...]ly loaded, 68 days.....	2 800	0.26	0.83	....	....
[...]y laid, 1 : 3 Portland-[...]ent mortar, 67 days...	2 920	0.27	0.87	2 870*	1.05
[...] laid, 1 : 5 Portland-[...]ent mortar, 65 days...	2 225	0.21	0.66	1 710	1.30
[...] laid, 1 : 3 natural-[...]ent mortar, 67 days...	1 750	0.16	0.52	305	5.75
[...]aid, 1 : 2 lime mortar, [...]days................	1 450	0.14	0.43	....	....
Underburned clay bricks					
[...] laid, 1 : 3 Portland-[...]ent mortar, 63 days...	1 060	0 27	0.31	2 870*	0.37

[...]age value based on thirteen tests of 1 : 3 Portland-cement mortar-cubes, 60 days

10 to 10½ ft. The lateral dimensions were 12½ by 12½ in. Two g of bricks were used, an excellent class of building bricks and a soft grade se as representative of inferior bricks. Different qualities of mortar and dif grades of workmanship were employed. Compression-tests of single gave these average results. For hard, shale building bricks, bedded in u crushing strength, flatwise, 10 700 lb per sq in; modulus of rupture, edg 6-in span, 1 670 lb per sq in. For soft or underburned clay bricks, cr strength, flatwise, 3 900 lb per sq in; modulus of rupture, 480 lb per The Macgregor tests showed that maximum strength with minimum e for brickwork is obtained with mortar made of ½ cu ft of Portland c ½ cu ft of hydrated lime and 3 cu ft of sand, or a 1 : 1 : 6 mixture.

Tensional Strength of Brickwork. See Chapter II, page 179.

2. Strength of Terra-Cotta and Terra-Cotta Piers

General Properties of Terra-Cotta. The lightness of terra-cotta, com with its great compressive strength, together with its relatively high res to the effects of heat and fire, renders it an especially valuable building ma Terra-cotta for building purposes, whether plain or ornamental, is ge made of hollow blocks formed with webs to give extra strength and ke work true while drying. This is necessary because good, well-burned, cotta cannot safely be made more than about 1½ in thick, whereas, w quired to bond with brickwork, it must be at least 4 in thick. When the cotta work does not project beyond the face of the wall these hollow spa generally filled with concrete or brickwork. For additional data regard fire-resisting properties and strength of ornamental and structural terr see Chapter XXIII, pages 814, 815, and 816.

Crushing Strength of Terra-Cotta Blocks. Some exhaustive expe made by the Royal Institute of British Architects give the following re the crushing strengths of terra-cotta blocks:

	Crushin per
Solid block of terra-cotta	523
Hollow block of terra-cotta, unfilled	186
Hollow block of terra-cotta, lightly made and unfilled	80

Tests of terra-cotta manufactured by a New York Company, whi made at the Stevens Institute of Technology in April, 1888 gave these

	Crushing weight per cu in		Crushin per
Terra-cotta block, 2-in square, red	6 840 lb	or	492
Terra-cotta block, 2-in square, buff	6 236 lb	or	449
Terra-cotta block, 2-in square, gray	5 126 lb	or	369

In tests for the New York Building Department, made at Columbia Un dense terra-cotta blocks developed a net crushing strength of 4 721 lb p or 340 tons per sq ft, and semiporous, 2 168 lb per sq in or 156 tons p these results being in each case the averages of a series of tests. (See p

From these results, the writer would place the safe working strength cotta blocks in the wall at 5 tons per sq ft when unfilled, and 10 tons when filled solid with brickwork or concrete.

...s of Terra-Cotta Piers. Tests * of terra-cotta block piers were made at the same time (January, 1907, and January, 1908) that the brick piers ...ed to in Table VII were made. The tests were made on terra-cotta piers, ...ngths of which varied from 9 ft 9 in to 12 ft 7¾ in. The lateral dimensions ...d from 8½ by 8½ in to 17½ by 17½ in. "The piers were built and tested ... lots, an interval of about one year separating the times of making the ... The two lots of piers were built of blocks which came in different ship... The cement used was the same brand in both years, although the lots

...e VIII. Tests of Terra-Cotta Piers, Made at the University of Illinois

...e amounts given are average values. The table gives results of tests of piers of ...d shipment, except for the concave-end blocks. The piers recorded in this table ...

...aracteristics of piers	... load lb per sq in	... to strength of block, gross area	... to strength of first of series	... mortar-cubes lb per sq in	... of pier to strength of cubes
... laid, 1 : 3 Portland-cement mortar, concentrically loaded	4 300*	0.83*	Standard 1.00*	......	1.26*
... laid, 1 : 3 Portland-cement mortar, eccentrically loaded	3 470	0.65	0.81*	3 090	1.12
...ly laid, 1 : 3 Portland-cement mortar, concentrically loaded	3 305	0.64	0.76	3 130	1.05
...rly laid, 1 : 3 Portland-cement mortar, eccentrically loaded	3 110	0.60	0.75	3 025	1.06
... laid, 1 : 3 Portland-cement mortar, concentrically loaded	3 050	0.59	0.71	3 370	0.88
... laid 1 : 5 Portland-cement mortar, concentrically loaded, inferior underburned blocks †	3 350	0.65	0.78	......	......
...ks with concave ends, 1 : 2 Portland-cement mortar	2 970	0.86	0.69	......	......

* Estimated. † Blocks of good quality, but underburned.

...e different. The terra-cotta block piers were generally made in sets of two. ... set was constructed and loaded similarly. Three of the piers were laid up ...ly (poorly laid); the remainder were built with the usual care given to ... work. The load was applied to the piers in different ways, although gen...ied continuously to failure." Some piers were loaded eccentrically ...nd one was loaded both concentrically and eccentrically, but the ...ccentric load was not sufficient to cause failure.

...tin No. 27, University of Illinois Engineering Experiment Station, Sept. 19...

Comparison of Results of Tests of Brick and Terra-Cotta Piers the tests summarized in Tables VII and VIII, "both the brick piers an terra-cotta block piers gave high strengths in all cases where strong morta care in building were used. The effect of the strength of the mortar was a ent in the carrying capacity developed in the piers, smaller loads being cated for piers built with 1 : 5 Portland-cement mortar than for those with Portland-cement mortar, and still smaller loads for those with 1 : 2 lime m The effect of the quality of the bricks is shown in the piers made with in bricks, these piers carrying only 31% as much as piers built with the l grade of bricks. In the case of the terra-cotta piers, the blocks which culled out as somewhat inferior gave a pier-strength which was perhaps less than the piers built with superior blocks. The effect of the attem represent hurried or careless workmanship in two brick piers and in three cotta block piers was a loss in strength of about 15% and 25% respective

"In the well-built brick piers, concentrically loaded, the ratio of streng pier to compressive strength of individual brick ranged from 31 to 37% in the underburned clay-brick pier the ratio was 27%. In the terra- block piers, concentrically loaded, the ratio of strength of pier to that of in ual block was 74% (an incompleted test) and 83, 85 and 89% for the o The higher ratio found for the terra-cotta block piers than for brick piers gests that the ability of individual pieces to resist transverse forces is a ment in the strength of the completed pier; and this suggestion may ha important bearing on the advantageous size of the component blocks may be used in a compression-piece where great strength is required.

"The strength of the pier is greater than that of the mortar-cubes in brick and terra-cotta block piers, except the soft-brick piers, which had b of low compressive strength. Both the strength of the individual bric blocks and the strength of the mortar affect the resistance of the pier, an relative effect of the two depends upon the character of the materials. evident, however, that the better the individual piece the more important to have a mortar of high resisting strength.

"The results obtained in applying the loads eccentrically were found to very well with those obtained from ordinary analysis.

"The quality of workmanship in laying up such columns has an impo bearing upon the resisting strength. The work of building piers, howev not difficult and requires only ordinary care. Full joints and an even be are important, and the ordinary workman ought to be able to construct of great strength. In the tests made on piers intended to represent po careless workmanship, the decrease in strength was not as much as anticip However, it must be understood that careful and trustworthy work is ess and that a few poor joints will materially reduce the strength of the stru Wherever good material and good workmanship are insured the streng masonry of this kind may be utilized with advantage."

Strength of Terra-Cotta Brackets or Consoles. A cornice-mod made by the Northwestern Terra-Cotta Company, 11½ in high at the wall 8 in wide on the face, and with a projection of 2 ft, was built into a wall an upper surface loaded with 2 tons of pig iron without any effect upon the modi Another bracket, 5½ in high, 6 in wide and with a 14-in projection, ma the East, broke at the wall-line under 2 650 lb, while a duplicate of it sust 2 400 lb for one month without breaking.*

The Weight of Terra-Cotta. The weight of terra-cotta in solid bloc 120 or 122 lb per cu ft. When made in hollow blocks 1½ in thick the w

* See The Brickbuilder, Vol. 7, page 142.

is from 65 to 85 lb per cu ft, the smaller pieces weighing the most. For 12 by 18 in or larger on the face, 70 lb per cu ft will probably be a fair ge. The tables in the manufacturers' catalogues give the various bearing-weights per square foot, thicknesses of parts, sizes of blocks, etc., for por- and semiporous blocks for all purposes.

2. Crushing Strength of Building Stones

(1) Sandstones

ngmeadow, Mass., Stone.* Reddish-brown sandstone, two blocks about 4 in in cross-section and 8 in in height.

ck No. 1 commenced to crack at 10 333 lb per sq in, and flew from the ne in fragments at 13 596 lb per sq in.

ck No. 2 commenced to crack at 3 012 lb per sq in and failed completely 121 lb per sq in.

dstone from Norcross Brothers' Quarries, East Longmeadow, Mass., Saulsbury Stone.* Block No. 1, 4 by 4 by 8 in high, commenced to crack 50 lb and failed at 8 812 lb per sq in.

ck No. 2, 4 by 4 by 8 in high, commenced to crack at 6 500 lb and failed 92 lb per sq in.

d Saulsbury Stone.* Block No. 1, 4 by 4 by 8 in high (about), commenced ck at 12 716 lb and failed at 13 520 lb per sq in.

ck No. 2, same size as No. 1, commenced to crack at 13 953 lb and failed 650 lb per sq in.

ite Stone.* Block No. 1, 6 by 6 by 6 in, commenced to crack at 12 590 lb failed at 12 619 lb per sq in.

ck No. 2, same size as No. 1, commenced to crack at 12 185 lb and failed 874 lb per sq in.

wn Stone from the Shaler & Hall Quarry Company, Portland, Conn.† sults of the tests are as follows:

Table IX. Crushing Strength of Brown Sandstone

Dimensions			Sectional area	First crack	Ultimate strength	Classification
[illegible] in	Compressed, surface in		sq in	lb	lb per sq in	
[illegible]	2.50	2.45	6.13	84 800	13 980	1st quality
[illegible]	2.48	2.47	6.13	81 700	13 330	1st quality
[illegible]	3 00	2.95	8.85	123 200	13 920	2d quality
[illegible]	2.98	2.97	8.85	122 000	15 020	3d quality
[illegible]	2.55	2.53	6.45	63 850	9 900	Bridge
[illegible]	2.48	2.52	6.25	58 340	9 330	Bridge

wn Stone from the Middlesex Quarry Company, Portland, Conn.‡ Four cubical blocks, about 1½ in square. Pressure per square inch at time of : No. 1, 10 928 lb; No. 2, 10 322 lb; No. 3, 8 252 lb and No. 4, 6 322 lb.

se tests were made with the United States testing-machines at Watertown l, Mass.

m tests made by Colt's Patent Fire-arms Manufacturing Company.

se tests were made with the United States testing-machines at Watertown l, Mass.

Red Sandstone * from Greenlee & Son's Quarries at Manitou, Col. specimen failed at 11 000 lb per sq in; weight, 140 lb per cu ft.

Light-Red Laminated Sandstone,† from St. Vrain Cañon, Col., a very stone, excellent for walks and foundations. Crushing strength on bed, 1 lb per sq in; weight, 150 lb per cu ft.

Gray Sandstone † (free-working) from Trinidad, Col. Crushing str 10 000 lb per sq in; weight, 145 lb per cu ft.

Gray Sandstone † from Fort Collins, Col. (laminated and similar in q to the St. Vrain stone). Crushing strength on bed, 11 700 lb per sq in; w 140 lb per cu ft. One ton of this stone measures just a perch in the wall.

(2) Granite

Red Granite † from Platte Cañon, Col. Crushing strength per square 14 600 lb; weight per cubic foot, 164 lb.

(3) Lava Stones

Lava Stone from the Kerr Quarries, near Salida, Col. Four cubical bl The results of the tests are as follows:

Table X. Crushing Strength of Lava Stone

Dimensions			Sectional area	First crack	Ultimate streng	
Height in	Compressed surface in		sq in	lb	lb	lb 1 sq
4.00	4.00	4.00	16.00	165 900	165 000	10
4.00	4.00	4.00	16.00	174 100	174 100	10
2.00	2.00	1.99	3.98	36 400	37 100	9
1.99	1.99	1.99	3.96	38 200	38 200	9

Lava Stone,‡ Curry's Quarry, Douglas County, Col. Crushing str 10 675 lb per sq in; weight, 119 lb per cu ft. Experience has shown th stone is not suitable for piers, or where any great strength is required cracks very easily.

(4) Marble and Limestone

White marble quarried at Sutherland Falls, Vt. Two cubical blocks 6 in square.§

Block No. 1 commenced to crack at 9 750 lb per sq in and failed sudde 11 250 lb per sq in.

Block No. 2 did not crack until it suddenly gave way at 10 243 lb per

Test of a Limestone Pier. A pier of Lemont limestone, 1 sq ft in cross-s and 9 ft in height, composed of seven stones with bearing surfaces plane fectly true and parallel to the natural bed and the joints washed with grout of the best English Portland cement, was tested at the Watertown A for William Sooy Smith, and only commenced to crack when the full of the machine, 400 tons, was exerted.

* These tests were made with the United States testing-machines at Wat Arsenal, Mass.

† From tests made for the Board of Capitol Managers of Colorado by State E E. S. Nettleton, in 1885, on 2-in cubes.

‡ From tests made by the Denver Society of Civil Engineers, in 1884, also cubes.

§ Tested at the United States Arsenal, Watertown, Mass.

(5) Bricks and Various Stones

Table XI gives the crushing strength of various kinds of bricks and building stones, the pressure being normal to the plane of the bed.

Table XI. Crushing Strength of Bricks and Stone *

Pressure at right-angles to bed

Kind of brick or stone	Crushing strength, lb per sq in
Bricks:	
Common, Massachusetts	10 000
Common, St. Louis, Mo.	6 417
Common, Washington, D. C.	7 370
Paving, Illinois	6 000 to 13 000
Granites:	
…e, Fox Island, Me.	14 875
Gray, Vinal Haven, Me.	13 000 to 18 000
Westerly, R. I.	15 000
Rockport and Quincy, Mass.	17 750
Milford, Conn.	22 600
Staten Island, N. Y.	22 250
East St. Cloud, Minn.	28 000
Gunnison, Col.	13 000
Red, Platte Cañon, Col.	14 600
Limestones:	
Glens Falls, N. Y.	11 475
Joliet, Ill.	12 775
Bedford, Ind.	6 000 to 10 000
Salem, Ind.	8 625
Red Wing, Minn.	23 000
Stillwater, Minn.	10 750
Sandstones:	
Dorchester, N. B. (brown)	9 150
Mary's Point, N. B. (fine grain, dark brown)	7 700
Connecticut brown stone,† (on bed)	7 000 to 13 000
Longmeadow, Mass. (reddish brown)	7 000 to 14 000
Longmeadow, Mass. (average, for good quality)	12 000
Little Falls, N. Y.	9 850
Medina, N. Y.	17 000
…den, N. Y. (red)	18 000 to 42 000
Cleveland, Ohio	6 800
North Amherst, Ohio	6 212
Berea, Ohio	8 000 to 10 000
Hummelstown, Pa.	12 810
Fond du Lac, Minn.	8 750
Fond du Lac, Wis.	6 237
…ton, Col. (light red)	6 000 to 11 000
St. Vrain, Col. (hard laminated)	11 505
Marbles:	
…r, Mass.	22 900
Rutland, Vt.	10 746
Montgomery Co., Pa.	10 000
Colton, Cal.	17 783
Italy	12 156
…ing:	
North River, N. Y.	13 425

* For more complete tables of the strength, weight and composition of building stones, see data, tables, etc. by Professor Thomas Nolan in Kidder's Building Construction and Superintendence, Part I, Masons' Work.

† This stone should not be set on edge.

(6) Additional Data on the Strength of Building Stones

Average Data for Building Stones of Good Quality. The following a relative values* are given by R. P. Miller.† SANDSTONE: weight, 150 cu ft; specific gravity, 2.40; crushing strength, 8 000 lb per sq in; sl strength, 1 500 lb per sq in; modulus of rupture, 1 200 lb per sq in; m of elasticity, 3 000 000 lb per sq in. GRANITE: weight, 170; specific g 2.72; crushing strength, 15 000; shearing strength, 2 000; modulus of r 1 500; modulus of elasticity, 7 000 000. LIMESTONE: weight, 170; s gravity, 2.72; crushing strength, 6 000; shearing strength, 1 000; mod rupture, 1 200; modulus of elasticity, 7 000 000. MARBLE: weight, 17 cific gravity, 2.72; crushing strength, 10 000; shearing strength, 1 400; ulus of rupture, 1 400; modulus of elasticity, 8 000 000. SLATE: weigh specific gravity, 2.80; crushing strength, 15 000; modulus of rupture, modulus of elasticity, 14 000 000. TRAP-ROCK: weight, 185; specific g 2.96; crushing strength, 20 000.

The following average relative values are given by A. I. Frye.‡ Th the results of tests made on small cubes of the materials. SANDSTONE: ing strength, 9 000 lb per sq in; GRANITE and GNEISS: crushing st 17 733 lb per sq in. LIMESTONES and MARBLES: crushing strength, 14 per sq in. SLATE: crushing strength, 10 000; ultimate tensional st 3 000; modulus of rupture, 5 000 lb per sq in.

When stones are not tested, Frye recommends the following average for ultimate strengths to be used in determining the safe stresses. SAND crushing strength, 5 000; ultimate tensional strength, 150; modulus of r 1 200 lb per sq in. GRANITE and GNEISS: crushing strength, 12 000; ulus of rupture, 1 600 lb per sq in. LIMESTONES and MARBLES: cr strength, 8 000; ultimate tensional strength, 800; modulus of rupture, 1 per sq in.

The following working unit stresses in pounds per square inch for ston or single blocks of stone are recommended by W. J. Douglass.§ SAND compression, 700; tension (direct and flexural), 75; shear,* 150. GR SYENITE and GNEISS: compression for hard, 1 500; for medium, 1 2 soft, 1 000; tension (direct and flexural), 150; shear, 200. LIMESTONE: pression for hard, 1 000; for medium, 800; for soft, 700; tension (dire flexural), 125; shear, 150. MARBLE: compression for hard, 900; for sof tension (direct and flexural), 125; shear, 150. BLUESTONE FLAGGING: co sion, 1 500; tension (direct and flexural), 200.

4. Compressive Strength of Mortars and Concretes

The Compressive Strength of Lime Mortar. The crushing stre common lime mortar, six months old and composed of 1 part lime to sand by measure, varies from 150 to 300 lb per sq in or from 10.8 to 21 per sq ft. Lime mortar alone should never be used where any but m loads are to bear upon the work, nor where the full loading is to be applied the mortar has had time to harden.

* The values in all cases are as follows: weight, in lb per cu ft; strength, mo rupture and modulus of elasticity, in lb per sq in.

† American Civil Engineers' Pocket Book (1912), page 357.

‡ Civil Engineers' Pocket-Book (1913), page 511.

§ American Civil Engineers' Pocket Book (1912), page 575.

Compressive Strength of Natural-Cement Mortar. The crushing gth* of natural-cement mortar, neat, averaged, for 7 days, 2 010; for s, 2 689; for 3 months, 3 646; and for 6 months, 5 052 lb per sq in. When with 2 parts of standard quartz sand, the mortar averaged in crushing th, for 7 days, 940; for 28 days, 1 390; for 3 months, 1 730; and for ths, 2 012 lb per sq in. For 2 years, an additional increase of 18% and y be assumed for the neat and sanded mortars, respectively, of natural nt.

Compressive Strength of Portland-Cement Mortar. The crushing gth* of Portland-cement mortar, neat, averaged, for 7 days, 5 915; for ys, 7 041; for 3 months, 7 347; and for 6 months, 9 760 lb per sq in. mixed with 3 parts of standard quartz sand, the mortar averaged, in ng strength, for 7 days, 941; for 28 days, 1 290; for 3 months, 1 490; r 6 months, 1 529 lb per sq in. When mixed with 3 parts of Ottawa sand, rtar averaged, in crushing strength, for 7 days, 1 199; for 28 days, 1 796; months, 1 887; and for 6 months, 2 181 lb per sq in. For 2 years, an nal increase of about 16% and 18% may be assumed for the neat and d mortars, respectively, of Portland cement.

tion of Compressive to Tensile Strength of Mortars. While it be stated as a very general guide that the compressive strength of hy-t-cement mortars is from six to ten times the tensile strength, these ratios iable and cannot be used as a reliable basis for calculations. The tensile th of Portland-cement mortars, under normal conditions, increases y during the first few days, the rate of change gradually falling off. In s the tensile strength is generally from one-half to two-thirds of the ulti-strength, which is practically reached in 2 or 3 months. The compressive th, however, continues to increase with age and the rate of increase varies ding to a somewhat different law.

Compressive Strength of Concrete. There are many reasons for riations in the values of the compressive strength of concrete and the pal factors are (1) the quality of the cement, (2) the size and character aggregates, (3) the quantity of the cement to a unit volume of the con-, (4) the manner of mixing, (5) the density of the mixture, (6) the condi-under which it seasons, and (7) its age; and of these various conditions ng the determination of the compressive strength the most important erally the proportions of the different ingredients of the mixture and its Although tables of average values of ultimate crushing strengths of con-are published and are of general value, they may be misleading unless dered with caution. In important operations it is advisable to have the ete tested and to adjust by trial the character and proportions of the in-nts until the required strength is obtained.

m of Specimen for Compression-Tests. For compression-tests of te in general, 4 to 12-in cubes of the mixture have been the standard of test-specimens; but since the advent of reinforced-concrete construc-nd the growth of the importance of determining the elastic properties of te, it has been found that a cylindrical test-specimen gives more definite s than a cube. A common shape of such cylinder is one in which the is about three times the diameter, and the cylinders are not less than 8 in. It is found that the compressive strengths of these cylinders of te are from 10 to 15% less than those of the cubes, but for cylinders of

* m compression-tests made by W. P. Taylor on cylindrical specimens 1 in in height, 1⅛ in in diameter and 1 sq in in cross-section.

still greater slenderness the compressive strengths remain about consta heights up to about seven diameters.

Compression-Tests on Concrete Cubes. From some tests made i for the Boston Elevated Railway Company at the Watertown Arsenal, o cubes of concrete made with five brands of Portland cement, coarse, sand and broken stone up to 2½-in size, having 49.5% voids, the fo average values of the compressive strengths were obtained:

Table XII. Compression-Tests on Concrete Cubes

Mixtures	7 days lb per sq in	1 month lb per sq in	3 months lb per sq in	6 months lb per sq in
1 : 2 : 4	1 560	2 400	2 900	3 820
1 : 3 : 6	1 310	2 160	2 520	3 090

Compression-Tests on Concrete-Cylinders. For cylindrical tes mens of concrete, made under reasonably good conditions as to chara materials and care in mixing, an average compressive strength of abou lb per sq in is usually developed in a 1 : 2 : 4 Portland-cement concrete 1 to 2 months; and of about 1 600 lb per sq in in a 1 : 3 : 6 mixture. the conditions are unusually favorable somewhat higher values than th obtained, but when the materials and workmanship are poor the ultima pressive stresses are lower.

Increase in Compressive Strength of Portland-Cement Concre regard to the increase of compressive strength of Portland-cement c with age, tests show that the ultimate compressive strength is nearly rea 60 days, at which time the strength varies from 80 to 90% of its value in time.

Ultimate Strengths of Natural-Cement Concrete. For natural concrete, the ultimate compressive, tensile and shearing strengths and t ulus of rupture may be taken at about one-half the corresponding va Portland-cement concrete, unless natural cements of known and teste are employed.

Strength of Unreinforced Concrete Columns. Short concrete c of lengths up to 10 or 15 diameters, develop a crushing strength of fro

Table XIII. Compression-Tests on Unreinforced Concrete Colum

Kind of concrete	Average age days	Average ultimate compressive stress lb per sq in
1 : 1 : 2	60	3 600*
1 : 1½ : 3	60	2 270
1 : 2 : 4	60	1 600
1 : 2½ : 5	60	1 200
1 : 3 : 6	60	935
1 : 3½ : 7	60	745
1 : 4 : 8	60	600

* This value was estimated as it was beyond the range of the tests.

less than that for short prismatic or cylindrical specimens. In Table XIII the results obtained by A. N. Talbot * on short, round, unreinforced stone-concrete columns, 12 in in diameter and 10 ft in length. A wet-mixture concrete used, of the different proportions shown, the forms were removed after days and the columns were tested through 60 days.

The values given in the table were deduced from the straight-line formula

$$\text{Ultimate compressive strength, lb per sq in} = \frac{12\,000}{S_a + S_t} - 400$$

which formula

S_a = the ratio of sand to cement
S_t = the ratio of stone to cement

example, in the 1 : 3 : 6 mixture, $S_a = 3$ and $S_t = 6$

Crushing Strength of Concrete Affected by Area of Bearing Surface. Professor Hool states † that if a load is applied over the central part, only, the bearing surface of a concrete test-specimen in compression, the unit load be greater than if it is applied over the entire surface; and this is due to the fact that the outer parts tend to assist the inner part to resist the stress. This shown by tests made on some of the 12-in concrete cubes used in the tests for the Boston Elevated Railway Company and referred to in the preceding paragraphs. Thirty-six of these concrete cubes were crushed by applying load over the entire upper bearing-surface of 144 sq in and an equal number similar concrete cubes were then crushed by applying the stress over a smaller 10 by 10 in, or 100 sq in. After this, the cubes of a third set were crushed the application of the stress over the still smaller area, 8 by 8¼ in, or 66 in. The tests of the second set gave unit crushing strengths 12% higher than first, and those of the third set unit crushing strengths 28% higher than first.

Working Stress for Bearing on Concrete. "When compression is applied surface of concrete of at least twice the loaded area, a stress of 32.5% of compressive strength may be allowed."‡

Working Stress for Axial Compression on Concrete. "For concentric compression on a plain concrete column or pier, the length of which does not exceed 12 diameters, 22.5 % of the compressive strength may be allowed." * (For the strength of reinforced-concrete columns, see Chapter XXIV, page 945.)

Recommended Ultimate Compressive Strengths of Portland-Cement Concrete.‡ Table XIV, of ultimate compressive strengths of concrete of different mixtures gives the values recommended by the American Society for Testing Materials, even though occasional tests show higher results. The values given are recommended as the maximum ultimate unit compressive strengths that should be used in design and on which the permissible working stresses should be based as a proper percentage of the same. The report referred states, also, that "in selecting the permissible working stresses to be allowed on concrete, we should be guided by the working stresses usually allowed for other materials of construction, so that all structures of the same class, but composed of different materials, may have approximately the same degree of safety." (For working stresses for concretes, masonry, etc., see this chapter, pages 265 to 276.)

* See University of Illinois Bulletin, No. 20, 1907, and Engineering News, Sept. 26,

† See Reinforced Concrete Construction, Vol I., page 18, by George A. Hool.

‡ Report of Committee on Concrete and Reinforced Concrete, of the American Society for Testing Materials, Nov. 20, 1912.

Table XIV. Ultimate Compressive Strengths of Different Mixtures of Portland-Cement Concretes

Aggregates	Mixtures				
	1 : 1 : 2 lb per sq in	1 : 1½ : 3 lb per sq in	1 : 2 : 4 lb per sq in	1 : 2½ : 5 lb per sq in	1 : 3 lb sq
Granite, trap-rock..........	3 300	2 800	2 200	1 800	1
Gravel, hard limestone and hard sandstone...........	3 000	2 500	2 000	1 600	1
Soft limestone and sandstone.....................	2 200	1 800	1 500	1 200	1
Cinders	800	700	600	500	

Effect of Consistency on the Crushing Strength of Concrete. Co that is mixed fairly dry and tamped until the moisture is brought to the su develops a somewhat greater compressive strength than concrete mixed more water. From a large number of tests * average compressive streng wet, plastic and dry concretes were determined. The age of the concret 1 year and 8 months, and five brands of cements were used. The mean pressive strengths were, for the wet concrete, 2 130; for the plastic, and for the dry, 2 350 lb per sq in.

In another series of tests † greater differences appeared. At the ag month the mean compressive strengths in pounds per square inch were, f wet concrete: granite, 3 155; gravel, 2 300; limestone, 4 195. For the m concrete: granite, 4 090; gravel, 3 545; limestone, 2 975. For the dam crete: granite, 4 520; gravel, 4 610; limestone, 4 365. At the end of 3 m the values for the granite aggregates were, for the wet concrete, 4 755; f medium, 4 990; and for the damp, 5 445.

Effect of Size of Stone on the Compressive Strength of Concrete may be stated, generally, that the use of stones of a maximum size cons with convenience generally results in a maximum compressive strength concrete. Stones of the larger sizes are generally more uniformly grade the smaller stones, and consequently grade better with the sand and give g strength. From tests ‡ made by W. B. Fuller, the average compressive stre at 140 days, of 1 : 9 concrete, were, for maximum size of stone ½ in., 1 per sq in; for 1-in stone, 1 150 lb per sq in; and for 2¼-in stone, 1 400 sq in.

Comparison of Compressive Strengths of Gravel and Stone Conc Concretes made with broken stone have, generally, a somewhat greate pressive strength than those made with gravel. From tests made by E. C the average compressive strength at 30 and 180 days, of concrete mad 1½-in maximum-size broken stone, was 20% greater than that of concrete of gravel of about the same size, the percentage of voids being nearly the 40% voids for the gravel and 47.4% voids for the broken stone. The a difference at 12 months, however, was reduced to 9%.

* Made for G. W. Rafter. See "Tests of Metals," 1898.
† Made in 1908. See Bulletin No. 344, United States Geological Survey.
‡ See Trans. Am. Soc. C. E., Vol. 59, 1907.

Effect of the Strength of the Aggregate on the Compressive Strength of Concrete. The compressive strength of trap-rocks, granites and most [illegible]stones is relatively so great that it cannot reduce the strength of the concrete itself. Some sandstones, however, have a much lower average compressive strength, and if they are friable and soft may lower relatively the final strength of the concrete. A concrete of low strength results from using cinders for the aggregate.

Building Laws for Working Loads on Masonry.* As previously mentioned (page 265) the building codes of most cities specify working loads to be used for masonry and as shown in Table II (page 267) these loads vary greatly. It is important, therefore, that the architect should be acquainted with the municipal [illegible] by which the construction of his building is governed. As building laws and regulations are constantly changing this information should be obtained from the code itself, care being taken that the latest edition and all supplements are consulted. A few requirements, peculiar to the codes in which they are [illegible], will be cited.

The Chicago code (1916) gives for eight classes of brickwork bearing values ranging from 100 lb per sq in for common brick with good lime mortar to 350 lb per sq in for paving brick with 1 to 3 Portland-cement mortar. This code discriminates between concrete mixed by hand and by machine, values of from [illegible] to 350 lb per sq in being given for hand-mixed concrete and from 300 to [illegible] per sq in for the same mixture if mixed by machine. The values in the [illegible] code are exceptionally low, common brick laid in lime mortar being [illegible] but 3 tons and concrete in foundations but 4 tons per sq ft. The [illegible]ville code introduces values for "Louisville-cement mortar." The practice [illegible] values of local material is to be commended. The Denver code gives [illegible] of 3 tons per sq ft on common brick with coal-dust in lime mortar, 3 tons [illegible] hollow tile in cement mortar and 8 tons for terra-cotta, solid, in cement. The Seattle code gives for the allowable compressive stress of mass concrete [illegible] of its compressive strength in twenty-eight days. The building code recommended by the National Board of Fire Underwriters is followed by a number of cities. This code includes in its list of allowable compression values, 1000 lb per sq in for Portland-cement grout, neat, and 1500 lb per sq in for Portland-cement grout, neat, not over ½ in thick, between steel members in foundations. For natural-cement concrete values are given of 125 lb per sq in for a 1 : 2 : 4 mixture and 80 lb per sq in for a 1 : 2 : 5 mixture. The average ultimate compressive strength for terra-cotta blocks designed to be normally laid with the cells vertical, and which are tested with the cells in that position, must not be less than 1200 lb per sq in. The allowable working stress on such blocks must not exceed 120 lb per sq in. The average ultimate compressive strength for terra-cotta blocks designed to be normally laid with the cells horizontal, and which are tested with the cells in that position, must not be less than 800 lb per sq in. The allowable working stress on such blocks must not exceed 80 lb per sq in. Hollow building blocks may be filled solidly with Portland-cement concrete or cement mortar to increase the stability and to aid in distributing the load, but the allowable working stress on such blocks must not be greater than that permitted for unfilled blocks.

* [illegible]ed from valuable data from Robins Fleming. See, also, pages 265 to 267 and Table II, page 267.

CHAPTER VI

FORCES AND MOMENTS

BY

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1. Composition and Resolution of Forces

Composition and Resolution of Forces. Imagine a round ball plac plane, frictionless surface at A (Fig. 1), the surface being perfectly level, the ball has no tendency to move until som is applied to it. If, now, the force, P, is to the ball in the direction indicated by the the ball will move in that direction. If, ins one force only, two forces, P and P_1, are ap the ball, it will not move in the direction o of the forces, but will move in the direction RESULTANT of these forces, or in the direct If the magnitudes of the forces P and indicated by the lengths of the lines, the complete the parallelogram $ABDC$, the d DA represents the direction and magnitu single force which has the same effect on as that resulting from the two forces P and P_1. If, in addition to forces P and P_1, the third force, P_2, is applied, the ball will move direction of the resultant of all three forces, and this resultant is obta completing the parallelogram $ADEF$, of which the resultant DA and th force P_2 are two adjacent sides. The diagonal R of this second parallelogram is the resultant of all three forces, and the ball will move in the direction Ae. In the same way the resultant of any number of forces may be found. Again, suppose a ball, whose weight is indicated by the length of the line W (Fig. 2), is suspended by two inclined cords. What are the magnitudes of the pulls or stresses which are developed in the cords and which keep the ball suspended at the point A? This is the converse of the last case. Instead of finding the diagonal or the resultant, the diagonal, which is the line W, is given, and the sides of the parallelogram are to be found. To find these the lines representing the directions of P and P_1 are prolonged and from B lines parallel to them are drawn to complete the parallelogram. Then CA is the required magnitude of the stress in cord P, and BC of that in cord P_1. Thus one fo have the same effect as many, or many the same effect as one.

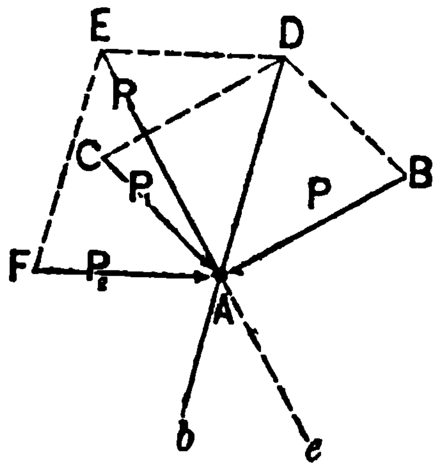


Fig. 1. Composition of Forces

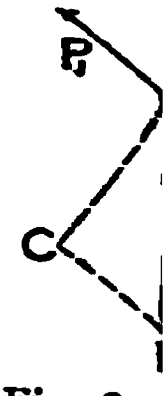
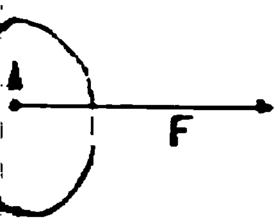


Fig. 2.
tion of

Forces Represented by Straight Lines. In considering the action o it is convenient to represent them graphically by straight lines with heads, as in Fig. 3. The length of the line, if drawn to a scale of pound sents the MAGNITUDE OF THE FORCE in pounds; the position of the line i

...OF ACTION; the arrow-head indicates its SENSE or the direction in which ...; and the point A its POINT OF APPLICATION. Thus the magnitude, direction and point of application are indicated and the force is completely represented.



...Force Represented ...a Straight Line

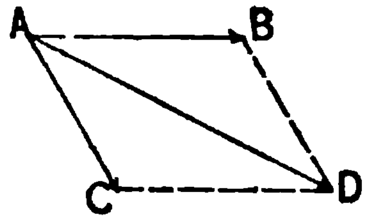


Fig. 4. Parallelogram of Forces

Parallelogram of Forces. If two forces applied at one point are represented in magnitude and direction by two ...t lines inclined to each other, their resultant is the diagonal of the ...ELLOGRAM formed on those lines. Thus, if the lines AB and AC (Fig. 4) ...nt two forces acting at a point A, to find the force which will have the ...effect as the two forces, the parallelogram ...C is completed and the diagonal AD drawn. ...line represents the RESULTANT of the two ... When the two given forces act at right... to each other, the magnitude of the resultant ...l to the square root of the sum of the ...s of the magnitudes of the other two forces.

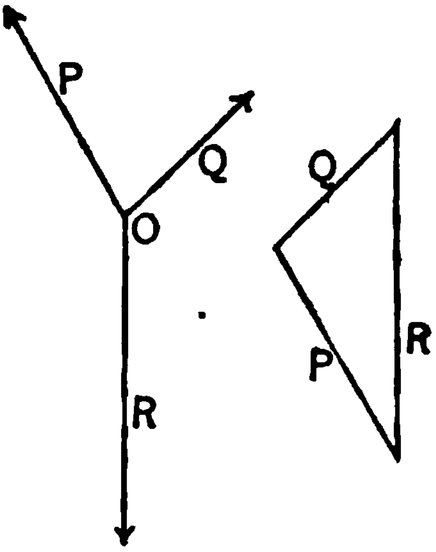


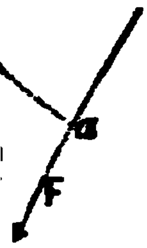
Fig. 5. Triangle of Forces

...ngle of Forces. If three forces acting at ...t are represented in magnitude and direction ... sides of a TRIANGLE taken in order, they are ...ibrium. Let P, Q and R (Fig. 5) represent ...forces acting at the point O. If a triangle ... drawn, like that shown at the right in Fig. ...ing sides respectively parallel to the direc... ...f the forces and taken in the same order, ...rces are in equilibrium. If such a triangle cannot be drawn, the forces ...t in equilibrium.

...**Polygon of Forces.** If any number of forces acting at a point can ...resented in magnitude and direction by the sides of a POLYGON taken ...er, they are in equilibrium. This follows directly from the preceding ...m.

2. Moments of Forces

...**ments.** In considering the stability of structures and the strength of ...ks, we are often obliged to take into consideration the moments of ...ces acting on a structure or on some part of a structure; and a knowledge of the general PRINCIPLES OF MOMENTS is essential to the proper understanding of these subjects. When we speak of the MOMENT OF A FORCE, we must have in mind some fixed point or line with respect to which the moment is taken. The moment of a force with respect to any given point, or CENTER OF MOMENTS, is the product of the magnitude of the force and the perpendicular distance from the point to the LINE OF ACTION of the force; or, in other words, the moment of a force is the product of the magnitude of the force by ... with which it acts. Thus if we have the force F (Fig. 6), and wish to ...ne its moment with respect to the point P, we determine the per...lar distance Pa, between the point and the line of action of the force, ...tiply it by the magnitude of the force. For example, if the magnitude



... Moment ...a Force

of the force F is 500 lb and the distance Pa is 2 in, the moment of t with respect to the point P is 500 lb × 2 in = 1 000 in-lb.*

Parallel Forces. If any body is in a state of rest or equilibrium u action of parallel forces, the sum of the forces acting in one direction eq sum of the forces acting in the direction. Thus if we have the forces P^1, P^2, P^3 and P^4 acting on AB (Fig. 7), in a direction opposite of the forces P_1, P_2 and P_3, then, if is in equilibrium, the sum of the fo P^2, P^3 and P^4 must equal the sum forces P_1, P_2 and P_3.

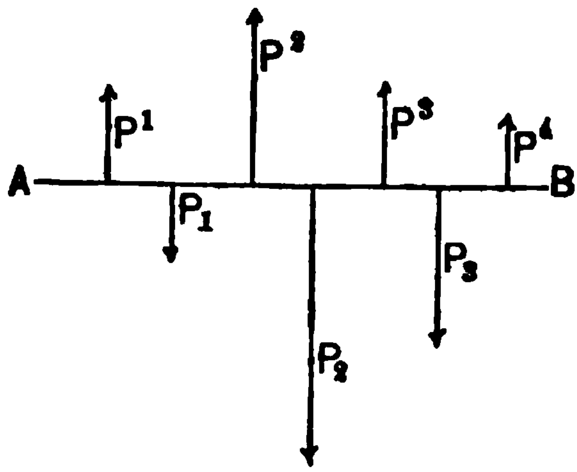


Fig. 7. Algebraic Sum of Unlike Parallel Forces

Parallel Forces Opposite in Ch If any number of parallel forces, acting in the same direction, act on if the body is in equilibrium, the sum moments of the forces tending to t body in one direction about an point is equal to the sum of the moments of the forces tending to tu the opposite direction. Let F_1, F_2 and F_3 (Fig. 8) represent three forces acting on the rod AB. If the rod is in equilibrium, the sum forces F_2 and F_3 is equal to F_1. Also, if we take the end of the rod, A, for the center of moments, the moment of F_1 is equal to the sum of the moments of F_2 and F_3 about that point, because the moment of F_1 measures the tendency to turn the rod CLOCKWISE, and the sum of the moments of F_2 and F_3 measure the tendency to turn the rod CONTRA-CLOCKWISE, and there is no more tendency to turn the rod one way than the other. For example, let the magnitude of forces F_2, F_3 each be represented by 5 force-units, the distance Aa by 2 length-units and the distance AB by 4 length-units. The magnitude of the force F_1 must equal the sum of the magnitudes of the forces F_2 and F_3, or 10 force-units, and its with respect to any point in the plane of the forces must equal of the moments of F_2 and F_3 with respect to the same point. If we t the center of moments, the moment of $F_3 = 5 \times 2 = 10$, and of $F_2 = 5$ › Their sum equals 30; hence the moment of F_1 must be 30. Dividing ment 30 by the force $F_1 = 10$ force-units, we have for the arm, 3 length- the force F_1 must act at a distance of 3 units from A to keep the rod in rium. If we take b as the center of moments, the force F_1 has no mo the length of its lever-arm is zero; and, for equilibrium, the moment of b must equal the moment of F_3 about the same point; or, as in this magnitudes of the forces F_2 and F_3 are equal, they must both be appli same distance from b, showing that b must be half-way between a and demonstrated before.

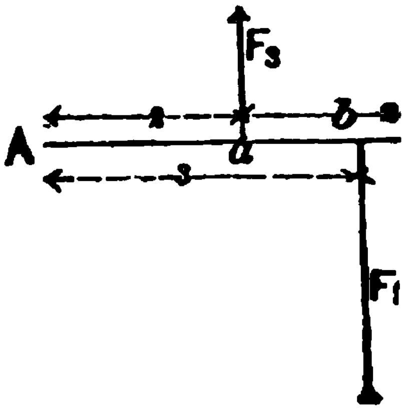


Fig. 8. Algebraic Sum ments of Unlike Paral

Three Parallel Forces. THE PRINCIPLE OF THE LEVER. This pri based upon the two preceding propositions and is of great importance

* The expressions POUND-FEET and POUND-INCHES are often given to thes to distinguish them from FOOT-POUNDS and INCH-POUNDS, by which WORK ar are measured.

...re. If a body is in equilibrium under the action of three parallel forces ...g in the same plane, each force is proportional to the normal distance be... the other two. Thus, if, as in Figs. 9, 10 and 11, three forces, P_1, P_2 and

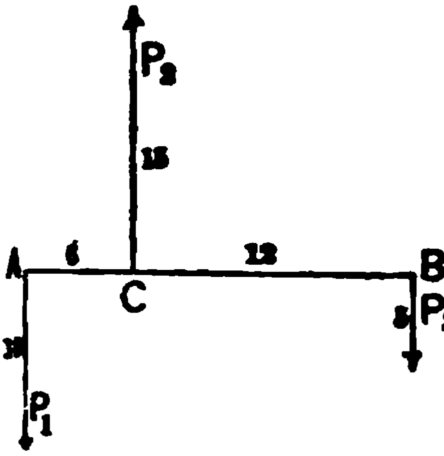


Fig. 9. Principle of the Lever

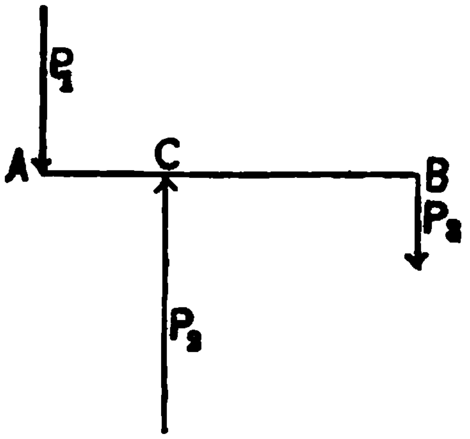


Fig. 10. Principle of the Lever

...t on the rod AB, in order that it may be in equilibrium, the following ...s must obtain between the magnitudes of the forces and the distances ...en their points of application;

$$\frac{P_1}{CB} : \frac{P_2}{AB} : \frac{P_3}{AC}$$

$$P_1 : P_2 : P_3 :: CB : AB : AC$$

...is the case of the COMMON LEVER and shows the method of determining ...weight a given lever will raise. The proportion is also true for any ar...ment of the forces (as shown in Figs. 9, 10 and 11), provided, of course, ...ces are lettered in the order shown in the ...s.

...example, let the distance AC be 6 in and the ...ce CB be 12 in. If a weight of 500 lb is ...d at the point B, how much will it raise at ...er end and what support will be required at ... 10)?

...ying the rule just given, we have the pro...:

$P_3 : P_1 :: AC : CB$ or $500 : P_1 :: 6 : 12$

$P_1 = 1\,000$ lb; or 500 lb applied at B will ...00 lb resting on or suspended at A. The ...ing force at C must, by the principles of PARALLEL FORCES IN ...RIUM, be equal to the sum of the forces P_1 and P_3, or 1 500 lb in ...e.

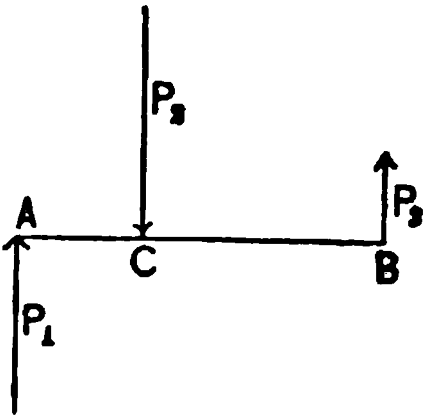


Fig. 11. Principle of the Lever

2. Center of Gravity

...al Principles. The LINES OF ACTION of the force of gravity converge ... the center of the earth; but the distance of the center of the earth from ...es which we have occasion to consider, compared with the size of those ...is so great, that we may consider the lines of action of the forces as ... The number of the forces of gravity acting upon a body may be con...as equal to the number of particles composing the body. The CENTER OF ... of a body may be defined as the point through which the resultant of ...lel forces of gravity, acting upon the body, passes for every position ...may. If a body is supported at its center of gravity and turned about

that point, it will remain in equilibrium in all positions. The resultant parallel forces of gravity acting upon a body is obviously equal to the w OF THE BODY; and if a force, equal in magnitude to the resultant, is a acting in a line passing through the center of gravity of the body, an direction opposite to that of the resultant, the body will be in equilibriu

Center of Gravity of a Straight Line. The word LINE here m material line whose transverse section is very small, such as a very fin The center of gravity of a straight line or rod of uniform size and mat at its middle point. This proposition is too evident to require demonst

The Center of Gravity of the Perimeter of a Triangle is at the of the circle inscribed in the triangle formed by the lines joining the points of the sides of the given triangle. let ABC (Fig. 12) be the given triangle. the center of gravity of its perimeter, fi middle points, D, E and F, and connect t straight lines. The center of the circle in in the triangle formed by these lines will center of gravity sought.

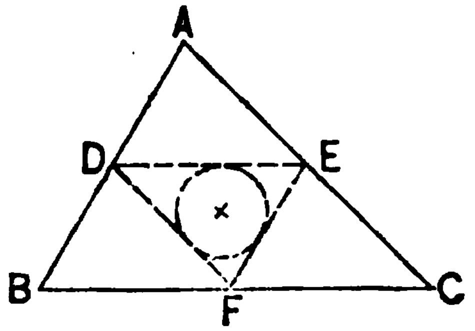


Fig. 12. Center of Gravity of Perimeter of Triangle

Center of Gravity of Symmetrical The center of gravity of a line which metrical with reference to a point is point. Thus the center of gravity of cumference of a circle or of an ellipse is at the geometrical center of the The center of gravity of the perimeter of an equilateral triangle, regular polygon, is at the center of the inscribed circle. The center of gr the perimeter of a square, rectangle, or parallelogram is at the interse the diagonals of those figures.

Center of Gravity of a Surface. A SURFACE here means a very th or shell. If a surface can be divided by a line into two symmetrical hal center of gravity will be on that line; if it can thus be divided by two li center of gravity will be at their intersection.

Center of Gravity of Regular Figures. The center of gravity of face of a circle or an ellipse is at the geometrical center of the figu equilateral triangle or regular polygon, at the center of the inscribed a parallelogram, at the intersection of the diagonals; of the surface of or of an ellipsoid of revolution, at the geometrical center of the body the convex surface of a right cylinder, at the middle point of the a cylinder.

Center of Gravity of Irregular Figures. Any figure bounded b lines may be divided into rectangles and triangles, and, the center of each part being found, the center of gravity of the whole figure may mined by treating the centers of gravity of the separate parts as parti weights are proportional to the areas of the parts they represent. (See

Center of Gravity of Triangles. To find the center of gravit angle, draw a line from each of two angles to the middle of the opp The intersection of the two lines is the center of gravity.

Center of Gravity of Quadrilaterals. To find the center of gra quadrilateral, draw the diagonals, and from that end of each diagon farthest from the intersection, lay off, toward the intersection, the its shorter segment. The two points thus formed, together with t

...ation, will form a triangle whose center of gravity is that of the quad-...al. Thus, let Fig. 13 be a quadri-...al whose center of gravity is to be ...d. Draw the diagonals AD and BC, ...from A lay off $AF = DE$, and from B ...off $BH = CE$. From E draw a line to ...middle of FH, and from F a line to the ...dle of EH. The point of intersection ...these two lines is the center of gravity ...the quadrilateral. This is a method ...monly used for finding the centers of ...vity of the voussoirs of an arch.

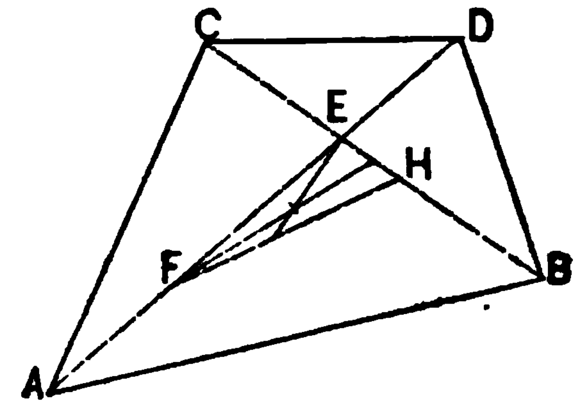


Fig. 13. Center of Gravity of Quadrilateral

...ble of Centers of Gravity. Let a ...a line drawn from the vertex of a figure ...the middle point of the base, and D the distance from the vertex to the center ...gravity of the figure. Then (Fig. 14):

In an isosceles triangle	$D = \frac{2}{3} a$
In a segment of a circle, vertex at center of circle	$D = \dfrac{\text{chord}^3}{12 \times \text{area}}$
In a sector of a circle, vertex at center of circle	$D = R \times \dfrac{2 \times \text{chord}}{3 \times \text{arc}}$
In a semicircle, vertex at center of circle	$D = \dfrac{4R}{3\pi} = 0.4244\,R$
In a quadrant of a circle	$D = \frac{3}{5} R$
In a semiellipse, vertex at center of circle	$D = 0.4244\,a$
In a parabola, vertex at intersection of axis with curve	$D = \frac{3}{5} a$
In a cone or pyramid	$D = \frac{3}{4} a$

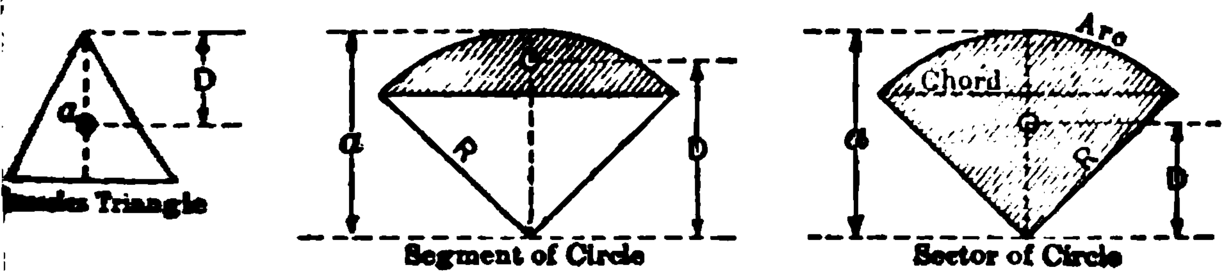
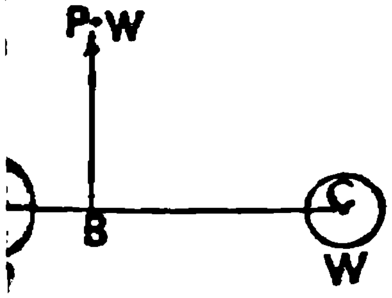


Fig. 14. Center of Gravity of Triangle, Segment and Sector

...frustum of a cone or pyramid, let h = the height of the complete cone or pyramid, h_1 = the height of the frustum, and let the vertex be at the apex of the complete cone or pyramid; then,

$$D = \frac{3\,(h^4 - h_1^4)}{4\,(h^3 - h_1^3)}$$



...15. Center of Gravity of ...Two Heavy Particles

Center of Gravity of Two Heavy Particles. Let P be the weight of a particle at A (Fig. 15), and W that of a particle at C. The center of gravity is at some point, B, on the line joining A ... The point B must be so situated that if the two particles were held ...by a stiff wire and supported at B by a force equal in magnitude to ...of P and W they would be in equilibrium. The problem then is

solved by the PRINCIPLE OF THE LEVER, and we have the proportion Three Parallel Forces. The Principle of the Lever),

$$P + W : P :: AC : BC$$

or

$$BC = \frac{P \times AC}{P + W}$$

If $W = P$, then $BC = AB$, or the center of gravity will be half-way betwee two particles. This problem is of great importance and has many pra applications.

Center of Gravity of Several Heavy Particles. Let W_1, W_2, W_3 and W_5 (Fig. 16) be the weights of the particles. Join W_1 and W_2 by a str line and find their center of gravity A, as i preceding problem. Join A with W_3 and fin center of gravity B, which will be the cent gravity of the three weights W_1, W_2, W_3. Pr in the same way with each weight. The last c of gravity found will be the center of gravity the particles. In both of these cases the lines jo the particles are supposed to be horizontal lin else the horizontal projections of the straight which join the points.

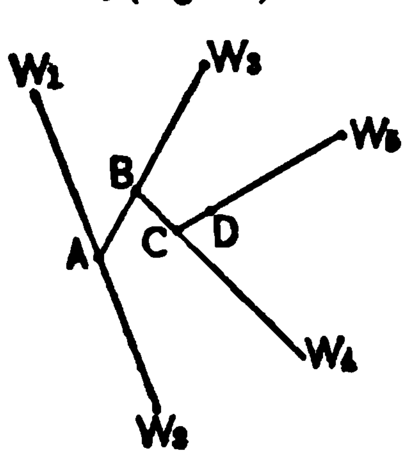


Fig. 16. Center of Gravity of Several Heavy Particles

Center of Gravity of Compound Sec Found by Moments. To determine the str of a beam having an unsymmetrical cross-secti is first necessary to determine the distance of the center of gravity of this s from the upper or lower surface of the beam. Various other computa also, involve finding the center of gravity of an irregular figure, so th problem is one of practical importance. If the figure of which the cen gravity is to be found can be divided into parts which are themselves r figures, the readiest and simplest method of finding the distance of the of gravity from one edge of the section is by means of MOMENTS. To explain this method assume a T-shaped section of uniform thickness, hinged on a wire XX, as in Fig. 17. The T section is made up of two rectangles, one forming the flange, the other the web. The center of gravity of each rectangle is at its own center of figure and may be readily found. If the T section is placed horizontally, as in the figure, the axis XX being fixed, it will immediately, by the force of gravity, revolve about the axis until it becomes vertical, and the sum of the moments of the forces causing the revolution is $A' \times d' + A'' \times d''$, A' representing the weight of the web and A'' the weight of the flange. To hold the T s in a horizontal position, there must be a moment of some force acting opposite, or upward, vertical direction and just equal to the sum two moments causing revolution downwards. If the force A, of this m tending to cause revolution upward, is equal to the weight of the entire tion, it must be applied at the center of gravity of the entire figure to m moment just equal to the sum of the moments of the two downward

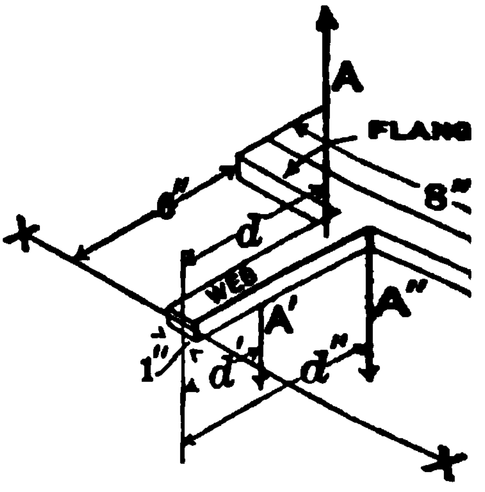


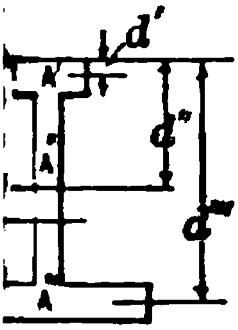
Fig. 17. Center of Grav Compound Sections by Mo

...moment of A is $A \times d$, therefore d is the distance from the end of the web, ...n the axis XX, to the center of gravity of the entire figure. Therefore, ...$A \times d = A' \times d' + A'' \times d''$,

$$d = \frac{A' \times d' + A'' \times d''}{A} \tag{1}$$

...weight of any homogeneous material of uniform thickness is proportional ...area, A, A' and A'' may be used to represent areas as well as weights. ...sing formula (1) as a rule, we have:

...ter of Gravity of Compound Figures. The distance of the center of ...y of a compound figure from any line of reference is equal to the sum of ...ducts, obtained by multiplying the area of each of the simple parts into ...the compound figure is divided by the distances of its center of gravity from the line of reference, divided by the area of the entire figure. This rule applies to any compound figure.



...8. Center of Gravity of Tees, Angles, ...nels, etc.

Example I. Assume that the T section shown in Fig. 17 has the dimensions indicated. Then A' equals 6, A'' equals 8, and A equals 14 sq in; and d' equals 3 and d'' equals $6\frac{1}{2}$ in. The sum of the products of A' by d' and ...d'' is 18 + 52 or 70 sq in × in, and this divided by 14 sq in, the area ...entire figure, gives 5 in for the distance d. The distance d of the center ...ity from the top of the webs, in each of the figures shown in Fig. 20, ...d by the following formula:

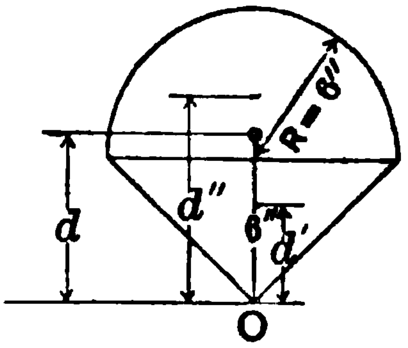


Fig. 19. Center of Gravity of Irregular I Section

$$d = \frac{\text{area of the web or webs} \times d'/2 + \text{area of flange} \times d''}{\text{area of the web or webs} + \text{area of flange}} \tag{2}$$

...section like that shown in Fig. 18, in which A', A'' and A''' represent the ...of the respective rectangles, the distance d of the center of gravity from ...p may be found by the formula

$$d = \frac{A' \times d' + A'' \times d'' + A''' \times d'''}{A' + A'' + A'''} \tag{3}$$

...ple II. To show the application of the rule for finding the center of ...y of compound figures, take the one shown in Fig. 19. The distance d

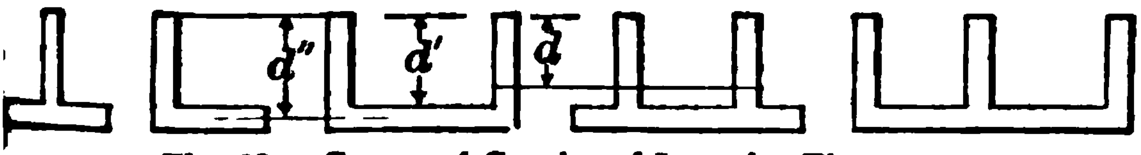


Fig. 20. Center of Gravity of Irregular Figures

...center of gravity of the entire figure from the vertex O is found as follows: ...a of the triangle is 36 sq in and of the semicircle 56.5 sq in. From the ...of Centers of Gravity (page 293) the distance of the center of gravity of ...this triangle from the vertex is two-thirds its height, which gives 4 in as

the value for d'. The center of gravity for a semicircle is 0.4244 R f
base, so that d'' equals 8.54 in. Then,

$$d = \frac{36 \times 4 + 56.5 \times 8.54}{36 + 56.5} = 6.77 \text{ in}$$

This method of finding the center of gravity is similar to that expla
Chapter IX for finding the supporting forces or reactions. In the latt
however, the problem is to find the balancing forces instead of the lever-

Additional Methods of Determining Graphically the Center of (of Irregular Plane Figures.* The center of gravity may be obtained ically by means of the FORCE-POLYGON and the EQUILIBRIUM-POLYGO figure or section considered, Fig. 21, is divided into parts whose cer

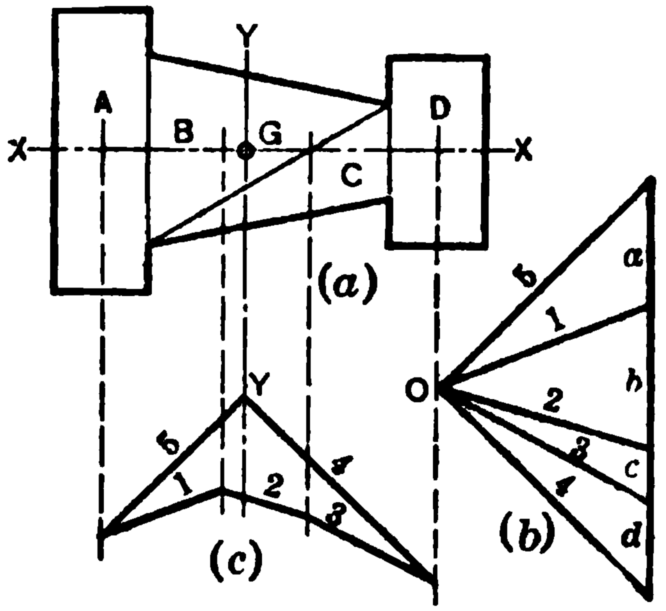


Fig. 21. Center of Gravity Determined Graphically.

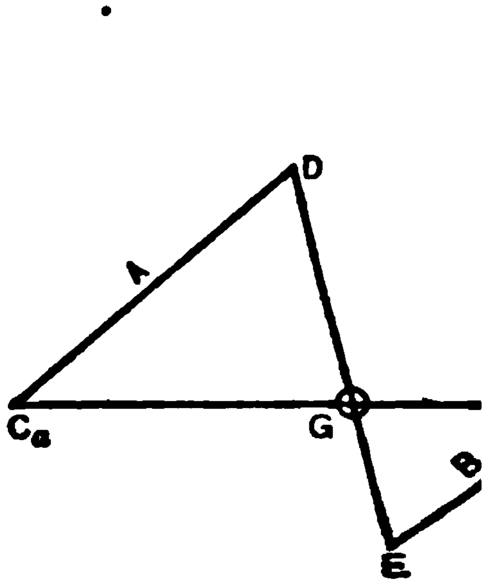


Fig. 22. Center of Gravity De Graphically. Second M

gravity can be located and areas calculated. The force-polygon (b) and rium-polygon (c) are drawn. The figure (a) is divided into rectangles angles, A, B, C and D, and vertical lines are drawn through their ce gravity. In (b) the vertical lines a, b, c and d are respectively proport length to the areas A, B, C and D. The pole, O, is located by the inters lines drawn at angles of 45° from the extremities of the line $abcd$. T 1, 2, 3, 4 and 5 are drawn from o, as shown, and the corresponding para drawn in (c). (See Figs. 3 and 5, pages 299 and 300; Figs. 12 to 1 314 to 320; and page 345.) The vertical line through Y, produced is a GRAVITY-AXIS and its intersection G with the horizontal gravity-axi the center of gravity of the figure. If the figure is not symmetric XX a second gravity-axis may be found by turning the figure through repeating the construction. The intersection of the two gravity-axes w center of gravity of the figure. ANOTHER METHOD is shown in Fig. the centers of gravity of two areas A and B be at the points C_a and C_b ively. From C_a the line C_aD is drawn in any direction, and its length on some given scale the area A. From C_b the line C_bE is drawn par and its length on the same scale represents the area B. The intersecti line joining D and E with C_aC_b is at the center of gravity of the areas For three areas A, B and C, the construction can be repeated by con and B as a single area; and so on for any number of areas.

* Condensed from data by Robins Fleming.

CHAPTER VII

STABILITY OF PIERS AND BUTTRESSES *

By
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LATE MEMBER OF AMERICAN INSTITUTE OF ARCHITECTS

...ical Principles. A pier or buttress may be considered STABLE when ...s acting upon it do not cause it to ROTATE OR TIP OVER nor cause any ...of masonry to SLIDE on its bed; some parts, however, of the masonry may ...ED. When a pier sustains a vertical load only, it might be con... STABLE, but it might not have sufficient STRENGTH. It is only when the ...ives a THRUST, as from a rafter or an arch, that its stability must be con...d. In order that there may be no rotation, ...ENT OF THE THRUST (Chapter VI) against ...r about any point in its outside edge must ...ed the MOMENT OF THE WEIGHT of the ...t the same point.

...strate let us consider the pier shown in ... Let us suppose that this pier receives the ... a rafter which exerts a THRUST T in the ... AB. The tendency of this thrust is to ...e pier to rotate about the outer edge b_1, ...e MOMENT OF THE THRUST about this point, ... is the measure of this tendency to rotate, ... $a'b_1$, $a'b_1$ being the lever-arm of the moment.

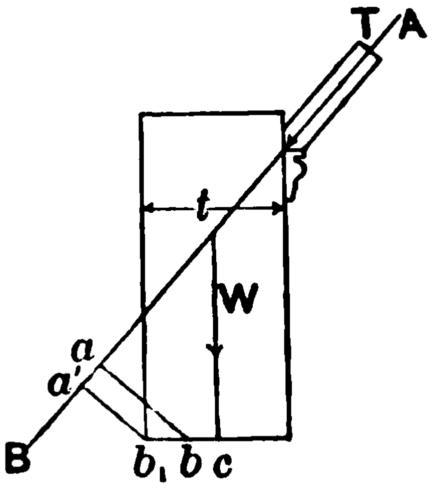

Fig. 1. Pier with Thrust

...TABLE EQUILIBRIUM, only, the MOMENT OF ...GHT of the pier about the same edge must just equal $T \times a'b_1$. The ... force representing the weight of the pier acts vertically through its ... of gravity which in this case is equidistant from its sides; and its lever-...e, or one-half its thickness.

..., for equilibrium of moments, we must have the equation

$$T \times a'b_1 = W \times b_1c$$

...n this condition the least additional thrust, or the crushing of the outer ...ll cause the pier to rotate; hence, for safety, we must use some FACTOR ...TY. This is sometimes done by making the moment of the weight equal ... of the thrust when referred to a point in the bottom of the pier, a cer-...tance in from the outer edge. This distance for piers or buttresses should ... less than one-fourth the thickness of the pier.

...enting this point in the figure by b, we have the necessary equation for ... stability of the pier

$$T \times ab = W \times t/4$$

...he width of the pier.

...not from this equation determine the dimensions of a pier to resist a ...rust, because we have the distance ab, t and W, all unknown quantities. ...e must first assume a tentative size for the pier, find the length of the ...nd see if the MOMENT OF THE WEIGHT of the pier is equal to the MO-... THE THRUST. If it is not we must assume another size for the pier. ...t of fact the steps of the problem usually present themselves in the

* See, also, Chapter XXXI, Section 2, Vaults.

inverse order, the pier or buttress being given and the determinati stability being required. The size of the pier or buttress is usually fi mined rather from the architectural exigencies of the design than engineering requirements for the stability of the structure. If upon i tion these are not in accord, it is the duty of the designers to use their i in seeing that both conditions are fufillled.

The Stability of Piers and Buttresses. When it is desired to d if a given pier or buttress is capable of resisting a given thrust, the pro be solved GRAPHICALLY in the following mann *ABCD* (Fig. 2) represent a pier which sustains thrust *T* at *B*. To determine whether the pier w sustain this thrust, we proceed as follows:

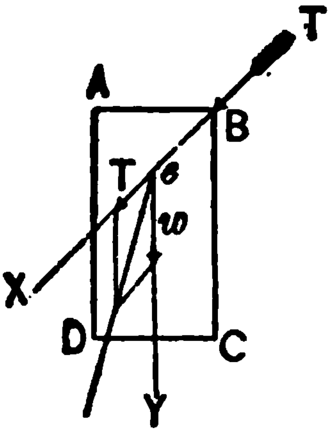

Fig. 2. Graphical Determination of Thrust on Pier

Draw the indefinite line *BX* in the directio thrust. Through the center of gravity of the pi in this case is midway between *AD* and *BC*, dra tical line intersecting the line of the thrust at force may be considered to act anywhere along i action, we may consider that the thrust and th act at the point *e*. The resultant of these two obtained by laying off the thrust *T* from *e* on *eX* weight of the pier *W*, from *e* on *eY*, both to the s of so many pounds to the inch, completing the ogram and drawing the diagonal. If this diagonal, prolonged, cuts of the pier at less than one-third the width of the base from t edge, the pier is generally considered unstable and its dimensions are (See Chapter IV, Theorem of the Middle Third.)

The Stability of Buttress with Offsets. THE STABILITY OF A PIE increased by adding to its weight by placing some heavy material o example, or by increasing its width at the base by means of OFFSE Fig. 3. Figs. 3 and 4 show the method of determining the stability tress with offsets. The first step is to find the vertical line passing th center of gravity of the whole pier. This is best done by dividing th into quadrilaterals, as *ABCD*, *DEFG* and *GHIK* (Fig. 3), finding the gravity of each quadrilateral by either the method of diagonals or tr explained in Chapter VI, and then measuring the perpendicular dist X_2, X_3 from the different centers of gravity to the line *KI*. (See, also VIII, page 313).

Multiply the area of each quadrilateral by the distance of its cent ity from the line *KI* and add together the areas and the products. I sum of the products by the sum of the areas and the result will be th of the center of gravity of the whole buttress from *KI*. This distance by X_0. This calculation is a practical application of the theorem in that the MOMENT OF THE RESULTANT of any number of forces abo point is equal to the SUM OF MOMENTS of the individual forces about

Example 1. Let the buttress shown in Fig. 3 have the dimensi Then the areas of the quadrilaterals and the distances from their gravity to *KI* are as follows:

First area	=	35 sq ft	$X_1 = 0'.95$	First area $\times$ X_1 =
Second area	=	23 sq ft	$X_2 = 2'.95$	Second area $\times$ X_2 =
Third area	=	11 sq ft	$X_3 = 4'.95$	Third area $\times$ X_3 =
		Total area, 69 sq ft		Total moments,

The sum of the moments of the areas is 155.55, and dividing this by the total , we have 2.25 as the distance X_8. Measuring this to the scale of the draw- from KI, we have a point through which the vertical line passing through the ter of gravity must pass.

The line, passing through the center of gravity of the buttress, can be found PHICALLY, also, by the method of the EQUILIBRIUM POLYGON (Fig. 3). (See

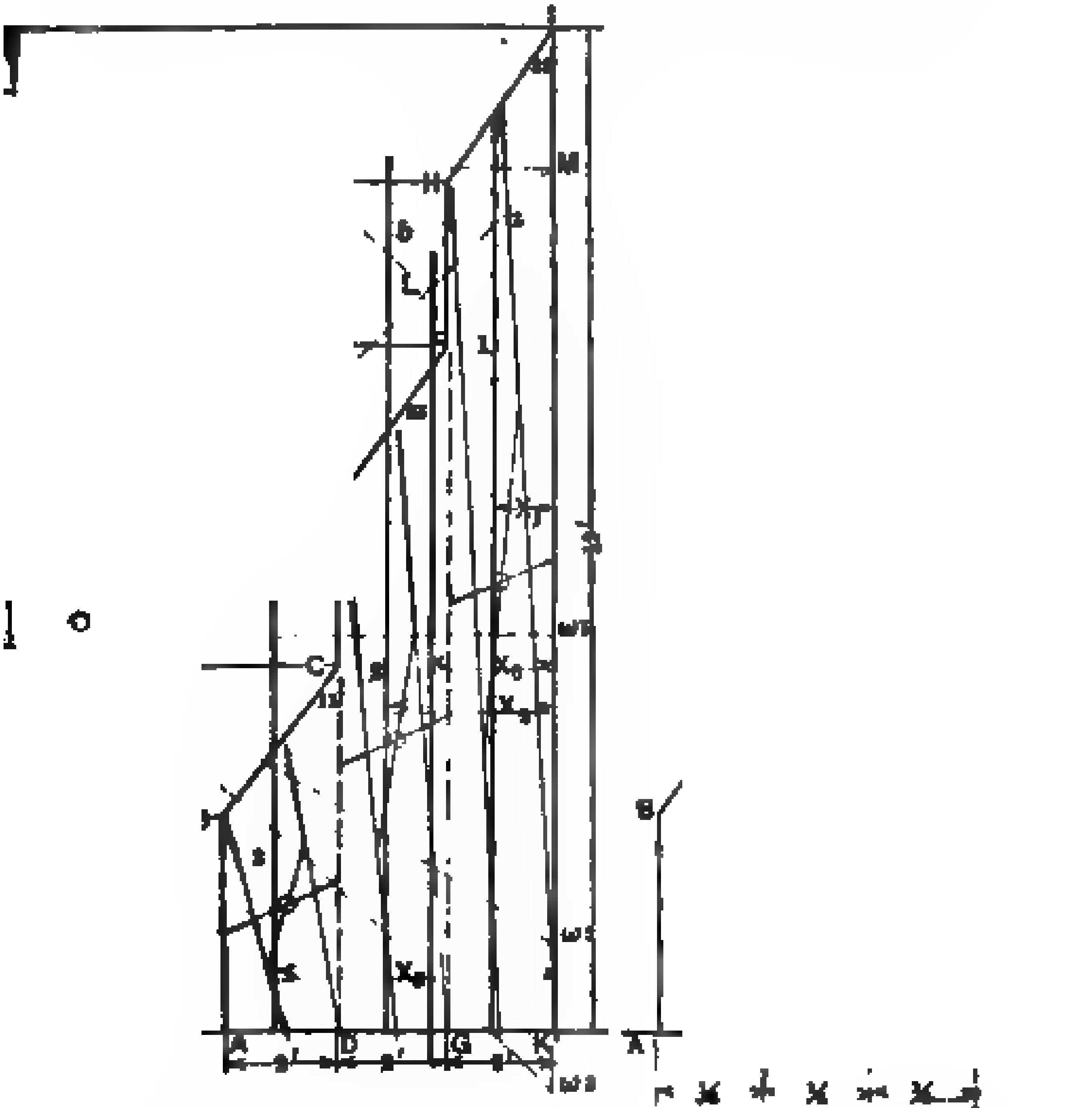

Fig. 3. Buttress with Offsets

Fig. 4. Resultant Thrust on Buttress with Offsets

296 and 314 to 320.) In order to do this, lay off at any convenient , beginning at some convenient point M, Mw 1, w 1 w 2, and w 2 w 3, the s of the various quadrilaterals composing the buttress. Through the center avity of each quadrilateral draw a vertical (green) line. Draw the lines and w 3 O, intersecting at some conveniently chosen POLE-POINT, O, Ow 1 and Ow 2. Through a, where MO intersects the vertical) line drawn through the center of gravity of the first quadrilateral, parallel to Ow 1, and through b, where ab intersects the (green) line the center of gravity of the second quadrilateral, draw bc parallel to Finally draw cL parallel to Ow 3. Where this line intersects MO be the point through which the (heavy red) line, passing through the ravity of the buttress taken as a whole, should be drawn. The distance

X_0, measured from IK, should then be 2.25 ft or very nearly this, allowi slight errors of drawing, and the same as that found by MOMENTS. Fig. 5 the same method of determining th tion of the center of gravity of a b similar to the one illustrated in Fig

After this line is found, the m of determining the stability of th is the same as that given for the p Fig. 2 and Fig. 4. If the butt more than one foot thick, that right-angles to the plane of the pa cubic contents must be determi order to find its weight. It is however, to divide the total thr the thickness of the buttress. Thi the thrust per foot of thickness buttress.

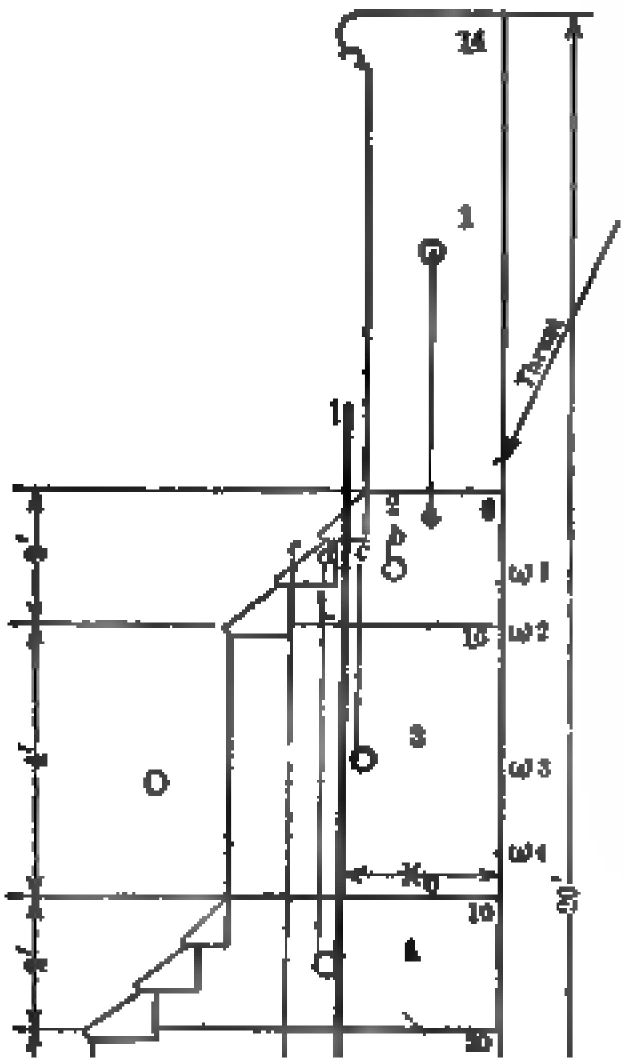

Fig. 5. Center of Gravity of Wall and Buttress

The Line of Pressure or L Resistance.* The LINE OF RESI or the LINE OF PRESSURE of a p buttress is a line drawn throu CENTER OF PRESSURE of each joint. The CENTER OF PRESSURE of any joint is the point in which the resultant of the forces acting on that portion of the pier above the joint cuts it. The line of pressure, or of resistance, when drawn in a pier, shows how near the greatest stress on any joint comes to the edges of that joint. It can be drawn by the following method.

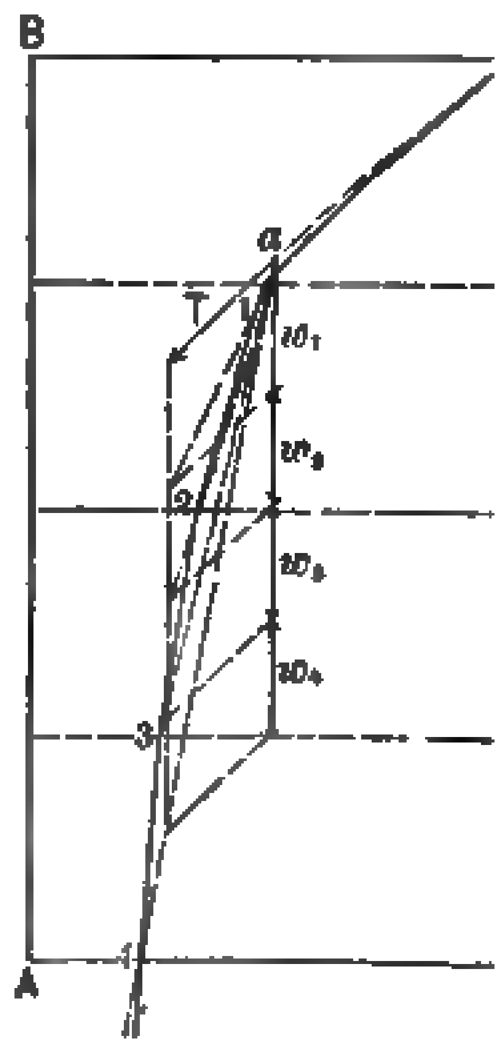

Fig. 6. Line of Pressure i

Let $ABCD$ (Fig. 6) be a pier whose LINE OF PRESSURE we wish to draw. Let T be the thrust against the pier divide the height of the pier into several parts, each 2 or 3 feet high, a by the horizontal dotted lines. It is more convenient to make the c

* This line is called, interchangeably, the line of pressure, the line of resist resistance-line, etc.

equal in height. Prolong the LINE OF THE THRUST, and draw a vertical through the center of gravity of the pier, intersecting the line of thrust at point *a*. From *a* lay off to a scale the thrust *T*, and the weights of the different parts of the pier, commencing with the weight of the upper portion. w_1 represents the weight of the portion above the first joint; w_2 represents the weight of the second part; and so on. The sum of the *w*'s will represent the weight of the whole pier.

Draw a parallelogram, with *T* and w_1 for its two sides. Draw the diagonal and produce it beyond the parallelogram, if necessary. Its point of intersection with the first joint will be a point in the line of pressure. Draw a second parallelogram, with *T* and $w_1 + w_2$ for its two sides. Draw the diagonal intersecting the second joint at the point 2. Continue in this way with the rest of the partial weights, the last diagonal intersecting the base *AD*, in the point 4. Join the points 1, 2, 3 and 4. The resulting broken line *C*1234 is the LINE OF PRESSURE or LINE OF RESISTANCE.

We have taken the simplest case as an example; but the same principles are true for any case. If the line of pressure of the pier at any point falls at a distance from the outside edge of the joint less than ONE-THIRD THE WIDTH OF THE JOINT, the pier is generally considered unsafe.

The Stability of a Wall and Buttress. By Moments and Graphical Method. The following example illustrates the application of these principles.

Example 2. Let Fig. 7 represent the section of a side wall of a church, with a buttress against it. Opposite the buttress, on the

Fig. 8. Stability of Wall and Buttress

a hammer-beam truss, which we will suppose exerts an wall of the church, amounting to about 9 600 lb. We at the resultant of the thrust acts at *P*, and at an angle tal. The dimensions of the wall and buttress are given ss is 2 ft thick, at right-angles with the plane of the ss the proper SIZE and FORM to enable the wall to resist

cide is whether or not the LINE OF PRESSURE cuts the ance in from *C* To ascertain this we must determine er of gravity of the wall and buttress above the joint *CD* determine this is by the METHOD OF MOMENTS, the s being taken about the line *KM* as an axis, or line of ready explained. The distance X_1 is, of course, half the r 1 ft. We next find the center of gravity of the part

CEFG (Fig. 8) by the method of diagonals; and scaling the distance X_2, ‑ it to be 2.95 ft.

The area $CEFG = A_2 =$ 10 sq ft; and the area $GIKL = A_1 =$ 26 sq ft $A =$ the total area above *CL*.

Then we have

$X_1 = 1$ ft	$A_1 = 26$ sq ft	$A_1 \times X_1 = 26$ sq ft × ft
$X_2 = 2.95$ ft	$A_2 = 10$ sq ft	$A_2 \times X_2 = 29.5$ sq ft × ft
	$A = 36$ sq ft	36)55.5 sq ft × ft
		$X_0 = 1.5$ ft

Expressed in EQUATIONS OF MOMENTS OF AREAS, this may be written as ‑ A representing the total area above the line *CL* (Fig. 8):

$$A \times X_0 = (A_1 \times X_1) + (A_2 \times X_2)$$

Hence,

$$X_0 = \frac{(A_1 \times X_1) + (A_2 \times X_2)}{A}$$

The center of gravity is at a distance 1.5 ft from the line *ED* (Fig. Fig. 7 measure the distance $X_0 = 1.5$ ft, and through the point *a* draw a line intersecting the line of the thrust prolonged at *O*. If the thrust is g for example, for a buttress 2 ft thick, it will be half that, or 4 800 l buttress 1 ft thick. We will call the weight of the masonry of which the l and wall is built, 150 lb per cu ft. Then the thrust is equivalent to 4 800/ cu ft of masonry. Laying this off to a scale from *O*, in the directio thrust, and the area of the masonry, 36 sq ft, from *O* on the vertical li pleting the rectangle, and drawing the diagonal, we find that the diago the joint *CD* at *t*, within the limits of safety. We must next find wl LINE OF PRESSURE cuts the base *AB*.

First, determine the position of the center of gravity of the whol This is determined by finding, as explained for the distances X_1 and X_2, tances X_2', X_3', in Fig. 8, and making the following computation, letti the total area above *AM*.

$X_1' = 1$ ft	$A_1' = 40$ sq ft	$A_1 \times X_1 = 40$ sq ft ×
$X_2' = 2.98$ ft	$A_2' = 24$ sq ft	$A_2 \times X_2 = 71.52$ sq ft ×
$X_3' = 4.95$ ft	$A_3' = 12$ sq ft	$A_3 \times X_3 = 59.40$ sq ft ×
	$A' = 76$ sq ft	76)170.92 sq ft ×
		$X_0' = 2.25$ ft

This, also, may be expressed in EQUATIONS OF MOMENTS OF AREAS, as e for the part above the line *CL*.

Then from the line *EB* (Fig. 7) lay off the distance $X_0' = 2.25$ ft, a through *d* a vertical line intersecting the line of the thrust at *O'*. On tical line, measure down from *O'* the whole area 76, to scale, as explaine and from the lower extremity of this line representing the area, lay o proper angle, the thrust $T = 32$. Draw the line *O'e*, intersecting the b This is the point where the LINE OF PRESSURE cuts the base; and, a a safe distance in from *A*, the buttress has sufficient stability. If th more offsets, we should proceed in the same way, finding where the PRESSURE cuts the joint at the top of each offset. The reason for doi

the LINE OF PRESSURE might cut the base at a safe distance from the edge, while higher up it might come outside of the buttress or too near outside face, thus making the buttress unstable. The method given in these ples is applicable to piers of any shape or material. If the LINE OF PRES- makes an angle of less than 30° with any horizontal joint, the stones the joint may SLIDE at this joint, or at least have a strong tendency to . SLIDING can be prevented either by doweling, or the joints. Such conditions, however, are in architectural construction.

Stability of a Wall and Buttress. Graphical od. This same example, which has been solved in foregoing case partly by MOMENTS and partly by ICAL METHODS, can be solved entirely by GRAPHICAL ODS. In this case it is not necessary to determine position of the line (the heavy red lines in Figs. 3 and sing through the center of gravity of the buttress as a whole. It is necessary, only, to determine (red) lines passing through the cen- of gravity of the various trapezoids rectangles into which it has been ided. To determine the position LINE OF PRESSURE and the various OF PRESSURE on the different , the method shown in Fig. 6 may ed. The construction shown in figure, in which the complete elograms of the forces acting at joint are drawn, may be simplified. If the parallelogram, only, or the ICLE OF THE FORCES acting at each , may be drawn and the whole uction placed at one side of the and afterwards transferred to the itself by means of parallel lines. the joint-planes *FG*, *EJ*, *CK* and calculate the areas of the parts of the wall and buttress, as *IKGF*, *FGJE*, *EJKC*, *CKNB* *NMA*, Fig. 9. These are respec- 14 sq ft, 6 sq ft, 16 sq ft, 10 sq ft sq ft. Lay off these areas to a of so many square units to a linear

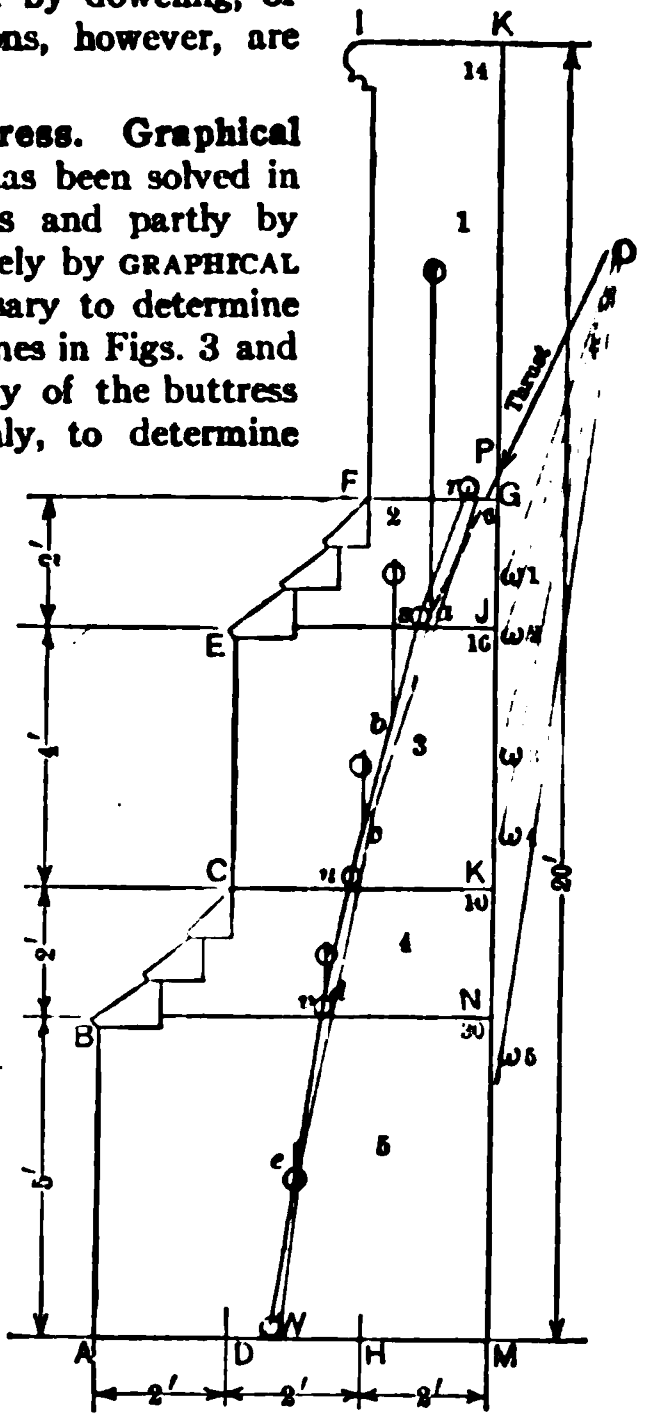

Fig. 9. Line of Pressure in Wall and Buttress

Pw 1, *w* 1 *w* 2, *w* 2 *w* 3, *w* 3 *w* 4 and *w* 4 *w* 5, along the line *KM*, begin- at the point of application of the THRUST. Lay off, at the same scale, the *OP* for one foot of thickness of the wall, and let this thrust be 4 800 lb. *Ow* 1, *Ow* 2, *Ow* 3, etc. Then *Ow* 1 will be the resultant of the thrust weight of the buttress above the joint *FG*, *Ow* 2 will be the resultant of resultant and the weight of that part of the buttress between the joints *EJ*, and so on until *Ow* 5 is reached, which is the resultant of the total of the buttress and the thrust as well as the resultant of the rectangle and the previous resultant. Prolong the thrust *OP*, until it cuts the) line through the center of gravity of the first rectangle *IKGF*, at *a*. his point draw a (green) line parallel to *Ow* 1 and prolong it backward

until it intersects the joint *FG* at the point within the small (red) circle. determines the CENTER OF PRESSURE on this joint. Next, draw *ab* (g parallel to *Ow* 2 and prolong it backward until it intersects the joint *EJ*, a CENTER OF PRESSURE on that joint. Repeat this operation to obtain CENTERS OF PRESSURE on each successive joint, drawing *bc*, *cd* and *de* pa respectively to *Ow* 3, *Ow* 4 and *Ow* 5.

It must be remembered, however, that *cd* does not have to be prolonged ward, as it cuts the joint *CK* below and to the left of the line passing thr the center of gravity of *EJKC*. Finally, join the various CENTERS OF PRES by the (red) broken line, which is the LINE OF PRESSURE in the buttress. this line lies within the MIDDLE THIRD of the construction, and the resulta the pressures on the various joint-planes do not make with the normals t joint-planes angles greater than the ANGLE OF FRICTION, the condition stability may be considered to be satisfied.

CHAPTER VIII

THE STABILITY OF MASONRY ARCHES *

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LATE MEMBER OF AMERICAN INSTITUTE OF ARCHITECTS

1. Arches

The Lintel and the Arch. When an opening is made in a masonry wall it is necessary to provide some means of spanning such opening to support the superimposed masonry. Two methods have been employed by constructors for this purpose. The first involves the use of the BEAM, GIRDER, CAP, or LINTEL, and the second the throwing of an ARCH from one side of the opening to the other. LINTELS are made of various materials, as wood, stone, reinforced concrete, cast iron and steel, and have cross-sections of different shapes. They are placed across the tops of the openings and transfer laterally the loads above, causing VERTICAL REACTIONS, only, in the side supports. An ARCH, on the contrary, is a particular arrangement of blocks of stone or other material, put together, generally along a curved line, in such a way that they resist the load by a balancing of certain THRUSTS and COUNTERTHRUSTS. An arch exerts on its supports an OUTWARD THRUST as well as a VERTICAL PRESSURE; and it is this outward thrust which requires that the arch should be used with caution when the abutments are not amply large and strong. The mechanical principles involved in the spanning of an opening by a lintel are much simpler than those of the arch and, historically, the lintel very considerably antedates the arch.

Definitions. Before taking up the principles of the arch, we will define the principal terms relating to it. The distance *ec* (Fig. 1) is called the SPAN of the arch; *ai*, the rise; *b*, the CROWN; the lower boundary *eac*, the SOFFIT or INTRADOS; the outer boundary *bb*, the BACK or EXTRADOS. The terms SOFFIT and BACK are also applied to the entire lower and upper curved surfaces of the whole arch. The sides of the arch which are seen are called the FACES. The blocks of which the arch itself is composed are called VOUSSOIRS; the center one, *K*, is called the KEYSTONE; and the lowest ones, *SS*, the SPRINGERS. In SEGMENTAL arches, or those of which the intrados is not a complete semicircle, the springers generally rest upon two stones, as *RR*, which have their upper surfaces cut to receive them; these stones are called SKEWBACKS. The line connecting the lower edges of the springers is called the SPRINGING-LINE; the sides of the arch are called the HAUNCHES; and the loads in the triangular spaces, between the haunches and a horizontal line drawn from the crown, are called the SPANDRELS. The blocks of masonry, or other material, which support two successive arches, are called PIERS; and the extreme blocks which, in the case of stone bridges, generally support, on one side, embankments of earth, are called ABUTMENTS. A pier strong enough to resist the thrust of one of two successive arches, in case the other one falls down, is some-

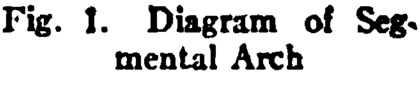


Fig. 1. Diagram of Segmental Arch

* See, also, Chapter XXXI.

times called an ABUTMENT-PIER. Besides their own weight, arches usuall port permanent loads or SURCHARGES of masonry or of earth.

Forms of Arches. In using arches in architectural construction FORMS of the arches are generally governed by the style of the building, a limited amount of space, rather than by engineering considerations. problem, therefore, that usually presents itself to the architect is not to the form and dimensions of an arch that will most economically and, fr engineering point of view, efficiently bear its load, but rather to determ an arch of a certain form and of certain dimensions will be stable an under its load. The SEMICIRCULAR and SEGMENTAL forms of arches a best as regards stability, and are the simplest to construct. ELLIPTICA THREE-CENTERED arches are not as strong as circular arches, and should be used where they can be given all the strength desirable.

The Strength of an Arch depends very much upon the care with whic built and upon the quality of the materials. In stone arches, special care s be taken to cut and lay the beds of stones accurately, and to make th joints thin and close, in order that the arches may be stressed as little as p in settling. To insure this, arches are sometimes built DRY, grout or mortar being afterwards run into the joints; but the advantage of this m is doubtful.

Brick Arches.* (See Figs. 2, 3, 4 and 5.) These may be built eit WEDGE-SHAPED bricks, molded or rubbed so as to fit to the radius of the

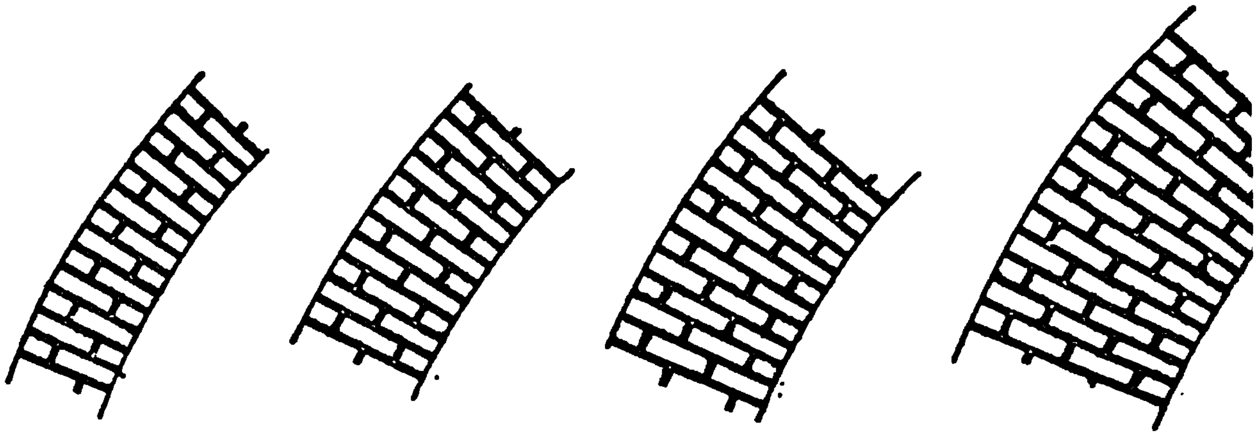


Fig. 2 Fig. 3 Fig. 4 Fig. 5
Brick Arches

or of bricks of COMMON SHAPE. The former method is undoubtedly the b it enables the bricks to be thoroughly bonded, as in a wall; but, as it in considerable expense to make the bricks of the proper shape, it is ver dom employed. When bricks of the ordinary shape are used, they a commodated to the curved figure of the arch by making the bed-joints th towards the intrados than they are at the extrados; or, if the curvature is s by driving thin pieces of slate into the outer edges of those joints; and dif methods are followed for BONDING them.

The usual method is to build the arch in concentric rings, each one-half thick; that is, to lay all the bricks as STRETCHERS and depend upon the t ity of the mortar for the connection of the several rings. Brick masonry structed in this way is deficient in strength, unless the bricks are laid in c mortar which is at least as tenacious as themselves. Another way is to i duce courses of HEADERS at intervals, and to connect pairs of half-brick

* For illustrations of the different methods of building brick arches, see Chapte Building Construction and Superintendence, Part I, Masons' Work, F. E. Kidder.

...ir. This may be done either by thickening with pieces of slate the ...of the outer ring of a pair of half-brick rings, so that there will be the ...number of courses of stretchers in each ring between two courses of ...ers; or by placing the courses of headers at such distances apart, that ...en each pair of them there will be one course of stretchers more in the ...than in the inner ring. The former method is best suited to arches of ...radius; the latter, to those of short radius. HOOP-IRON laid around the ...between half-brick rings, as well as longitudinally and radially, is very ...for strengthening brick arches. The bands of hoop-iron which traverse ...ch radially may also be bent, and prolonged into the bed-joints of the ...g and spandrels. By the aid of HOOP-IRON BOND, Sir Marc-Isambard ...el built a half-arch of bricks, laid in strong cement mortar, which stood, ...ting from its abutment like a bracket to the distance of 60 ft, until it was ...oyed by the undermining of its foundations.

...e only requirements in the New York City Building Laws for brick and ...arches is that "openings for doors and windows in all buildings shall have ...and sufficient arches of stone, brick, or terra-cotta, well built and keyed ...with good and sufficient abutments."

...e for the Radius of Brick Arches. A good RULE for the radius of ...ntal brick arches over windows, doors and other small openings is to ...e the RADIUS EQUAL TO THE ...H OF THE OPENING. This ...a good rise to the arch and ...sing proportion. In com... brickwork, when no par...r architectural effect is ...d, such as in the rowlock ...s thrown over the openings ...ilar walls, a RULE in very ...on use is to make the RISE ...e arch at the crown AN INCH ...IGHT FOR EVERY FOOT OF ...

...gmental Arches with Tie-... It is often desirable to ...openings in a wall by means ...ches when there are not a ...ient number of abutments ...thstand the thrusts. In ...of this kind each arch can be sprung from two cast-iron SKEWBACKS, held ...er by IRON RODS as is shown in Fig. 6. When this is done, it is necessary ...portion the size of the rods to the THRUST of the arch. The HORIZONTAL ...T of the arch may be very closely determined by the following formula:

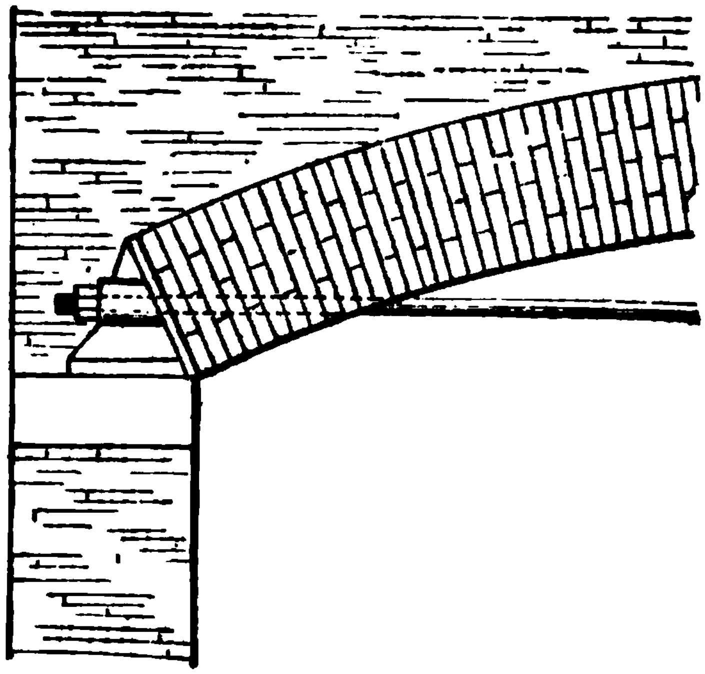


Fig. 6. Segmental Brick Arch, Cast-iron Skew-back and Wrought-iron Tie-rod

$$\text{Horizontal thrust} = \frac{\text{load on arch} \times \text{span}}{8 \times \text{rise of arch in feet}}$$

...he load is concentrated at the center of the arch, the thrust will be twice ...ven by this formula.

...TENSIONAL STRESS in the rod or rods will equal the HORIZONTAL THRUST ...arch and if there are two rods, the stress in each will be one-half the .... If there are three rods, then each must resist one-third the thrust. ...ng the stresses in the rods, the size of each may be determined from ...II, Chapter XI.

Example 1. Let us assume that a brick arch, like the one shown in] has a span of 15 ft, a rise at the center of 1 ft 6 in, and that it supports a brick wall. The weight of all the brick masonry above the arch does not upon it. Usually only an EQUILATERAL TRIANGLE of brickwork is consi the base of the triangle being the span. Assume, therefore, an equilater angle the sides of which are each 15 ft long. The altitude of this tria about 12.6 ft and its area will equal 15 ft × 12.6 ft × ½ = 94½ sq ft. If t is 12 in thick there will be 97½ cu ft of brickwork within this triangle wall; and since ordinary brickwork weighs about 115 lb per cu ft, its will be about 10 867 lb. Substituting these values in the formula,

$$\text{The horizontal thrust} = \frac{10\,867 \times 15}{8 \times 1.5} = 13\,584 \text{ lb}$$

Looking in Table II, page 388, it appears that one 1½-in or two 1⅛-in round, wrought-iron rods, or one 1⅛-in or two ¾-in round, upset, ste should be used.

Centers for Arches. A CENTER is a temporary structure, gener timber, on which the voussoirs of an arch are supported while the arch i built. It consists of parallel frames or ribs, placed at convenient di apart, curved on the outside to a line parallel to that of the soffit of th and supporting series of transverse planks, upon which the arch-stone The center commonly used is one which can be lowered, or STRUCK all piece, by driving out wedges from below it, so as to remove at once the s from every point of the arch. The center of an arch should not be struc the solid part of the backing has been built and the mortar has had t set and harden; and when an arch forms one of a series of arches wit between them, no center should be struck so as to leave a pier with a abutting against one side of it only, unless the pier has sufficient stability as an abutment. When possible, the STRIKING of the center of large brick should be delayed for two or three months after the arch is built, and the period that they are in place they should be EASED from time t This is done by EASING OUT the wedges under the centers a little at a as to let them down gradually and thus adjust any slight settling or shrin the masonry as it occurs.

Mechanical Principles of the Arch. In designing an arch, the firs tion to be settled is the FORM of the arch; and in regard to this, as already there is generally little choice. When the abutments are of ample size, MENTAL ARCH is the strongest; but when it is necessary to make the ab of the arch as small as possible, the SEMICIRCULAR or the POINTED ARCH be used.

Depth of Keystone. Having decided upon the form of the arch, the OF THE ARCH-RING must next be decided. This is generally determined puting the required DEPTH OF THE KEYSTONE and making the depth whole ring the same or a little larger. In considering the strength of a the depth of the keystone is considered to be only the distance from trados to the intrados of the arch; and if the keystone projects above t ring, as in Fig. 1, the projection is considered a part of the load on t There are several rules for determining the depth of the keystone, but empirical; and they differ so greatly that it is difficult to recommend a ticular one.

Rankine's Formula for Depth of Keystone. Professor Rankin is often quoted, and gives results which are probably true enough f

...is. It applies to both CIRCULAR and ELLIPTICAL ARCHES and is as follows. ...is a mean proportional between the inside radius at the crown, and 0.12 of ...t for a single arch, and 0.17 of a foot for an arch forming one of a series:

$$\text{Depth in feet of keystone for single arch} = \sqrt{(0.12 \times \text{radius at crown})}$$

$$\text{Depth in feet of keystone for arch of a series} = \sqrt{(0.17 \times \text{radius at crown})}$$

...e dimensions given by this formula seem to agree very well with those ...ally used in practice in arches of a certain kind. The formula, however, ...s the same depth of keystone for spans of any length, provided the radius ...e same; and in this particular it would seem that the rule is not satisfactory.

...rautwine's Formula for Depth of Keystone. Trautwine, from calcu...ns made for a large number of arches, deduced a formula for the depth of ...tone, which seems to agree with theory more closely than Rankine's formula. ...rule is, for CUT STONE,

$$\text{Depth of key in feet} = \left(\frac{\sqrt{\text{radius} + \text{half span}}}{4}\right) + 0.2 \text{ ft}$$

...or SECOND-CLASS work this depth may be increased about one-eighth part, ...r BRICKWORK OR FAIR RUBBLE, about one-third.

...ables for Depths of Keystones. Table I gives a few examples of the ...HS OF THE KEYSTONES of some bridges, together with the depths which ...ld be required by Trautwine's or Rankine's formula. From this table it ...ms that the results of both formulas agree very well with dimensions used ...tual practice.

Table I. Depths of Keystones of Some Arches of Circular Arc

...ame or location of structure	Span	Rise	Radius	Actual depth of key	Calculated depth of key: Trautwine's Rule	Calculated depth of key: Rankine's Rule	Engineer
	ft	ft	ft	ft	ft	ft	
...in John, Washing-...n aqueduct........	220.0	57.25	134.25	4.16	4.11	4.00	Meigs
...svenor bridge ...hester, England....	200.0	42.00	140.00	4.00	4.07	4.10	Hartley
...a Riparia, Turin, ...aly	148.0	18.00	160.10	4.92	4.03	4.38	Mosca
...eland, England.	118.0	38.00	64.80	3.50	3.00	2.79	Telford
...n bridge, Scotland, ...a series..........	90.0	[illegible]	48.90	3.00	2.62	2.88	Telford
...s bridge, Phila-...phia & Reading ...ilroad..........	78.0	25.00	43.00	3.00	2.46	2.27	Steele
...nt St. bridge, ...iladelphia, brick ...ement..........	60.0	18.00	34.00	2.50	2.20	2.00*	Kneass
...delphia & Read-...g Railroad........	44.0	8.00	34.30	2.50	2.08	2.02	Steele
...delphia & Read-...g Railroad........	31.2	5.00	26.80	1.66	1.83	1.79	Steele

* For first-class cut-stone work.

Table II* gives the DEPTHS OF KEYSTONES for arches of first-class cut st according to Trautwine's Formula. For second-class cut stone, add about eighth part and for fair rubble or for brickwork about one-third part, as st with formula.

Table II. Depths of Keystones for Arches of First-Class Cut-Stone Masonry

Span	Rise, in parts of the span						
	1/2	1/3	1/4	1/5	1/6	1/8	1/1
ft	ft	ft	ft	ft	ft	ft	f
2	0.55	0.56	0.58	0.60	0.61	0.64	0.
4	0.70	0.72	0.74	0.76	0.79	0.83	0.
6	0.81	0.83	0.86	0.89	0.92	0.97	1.
8	0.91	0.93	0.96	1.00	1.03	1.09	1.
10	0.99	1.01	1.04	1.07	1.11	1.18	1.
15	1.17	1.19	1.22	1.26	1.30	1.40	1.
20	1.32	1.35	1.38	1.43	1.48	1.59	1.
25	1.45	1.48	1.53	1.58	1.64	1.76	1.
30	1.57	1.60	1.65	1.71	1.78	1.91	2.
35	1.68	1.70	1.76	1.83	1.90	2.04	2.
40	1.78	1.81	1.88	1.95	2.03	2.18	2.
50	1.97	2.00	2.08	2.16	2.25	2.41	2.
60	2.14	2.18	2.26	2.35	2.44	2.62	2.
80	2.44	2.49	2.58	2.68	2.78	2.98	3.
100	2.70	2.75	2.86	2.97	3.09	3.32	3
120	2.94	2.99	3.10	3.22	3.35	3.61	3.
140	3.16	3.21	3.33	3.46	3.60	3.87	4
160	3.36	3.44	3.58	3.72	3.87	4.17	...
180	3.56	3.63	3.75	3.90	4.06	4.38	...
200	3.74	3.81	3.95	4.12	4.29	....	...
220	3.91	4.00	4.13	4.30	4.48	....	...
240	4.07	4.15	4.30	4.48	....	....	...
260	4.23	4.31	4.47	4.66	....	....	...
280	4.38	4.46	4.63	....	....	....	...
300	4.53	4.62	4.80	....	....	....	...

Example 2. Having decided what the thickness of the arch-ring will remains to determine whether such an arch would be stable if built. following example will illustrate the method of determining this.

Consider an unloaded semicircular arch of 20-ft span.

First, to find the depth of the keystone, we will use Rankine's Formul

$$\text{Depth of key} = \sqrt{0.12 \times 10} = \sqrt{1.2} = 1.1 \text{ ft}$$

Trautwine's Formula gives nearly the same result,

$$\text{Depth of key} = \frac{\sqrt{10 + 10}}{4} + 0.2 \text{ ft} = 1.3 \text{ ft}$$

But if we should compute the stability of a 20-ft semicircular arch keystone 1.3 ft deep, we should find that the arch is very unstable; h this case, we cannot use the formula and must act upon our own jud In the opinion of the author, the arch-ring of such an arch should be 2½ ft deep and the stability of the arch should be tested for that th In all calculations on the arch, it is customary to consider it 1 ft t

* Taken from The Civil Engineer's Pocket-Book, John C. Trautwine.

...angles to its face. This allows the AREAS OF THE FACES to be substituted ...the ACTUAL WEIGHTS of the voussoirs and their loads. This method was ...d in the discussion of Retaining-Walls, Chapter IV, and Piers and But...ts, Chapter VII. Furthermore, it is evident that if an arch 1 ft thick is ...ble, any number of arches of the same dimensions built alongside of it would ...stable. In determining the stability of masonry arches it is also customary ...eglect any increase in the strength of the arch from the mortar in the joints, ...in other words, to consider the arch as laid up dry.

...aphic Determination of the Stability of Arches. An arch has already ...n defined as a particular arrangement of blocks of stone or other material, ...e blocks being called the vous...rs. For the sake of simplicity ...ider an UNLOADED ARCH. In ...h an arch each voussoir is sub...ted to the action of three forces, ...) the thrust that it receives from ...e voussoir next above it in the ...h-ring, (2) the force of gravitation, ...ts own weight and (3) the reaction ...the resultant thrust. The first two ...es combine into one and form ...thrust that this voussoir exerts on the one next below it in the arch-ring ...g. 7). The points in which these various thrusts cut the joints are called the ...ERS OF PRESSURE of the joints, while the line joining these centers of pres...e is called the LINE OF PRESSURE or LINE OF RESISTANCE.* In order that an ...h may be absolutely stable, this line of resistance must fall within the MIDDLE ...D of the arch-ring. (See Theorem of the Middle Third, Chapter IV) If ...arch is stable the centers of pressure on the various joint-lines are within ...middle third of the voussoir-depths and the angles made by the different ...sts with the normals to the joints are less than the ANGLE OF FRICTION of ...material of which the arch is constructed. If these conditions are not ful-

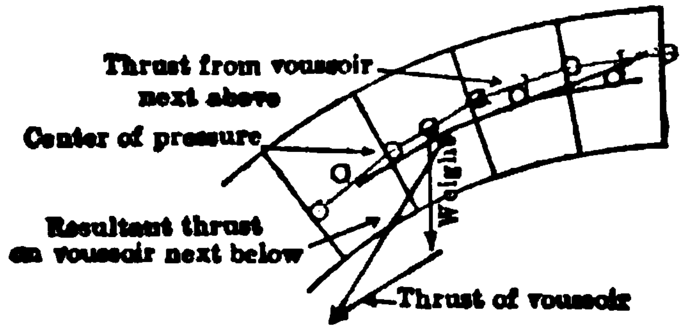
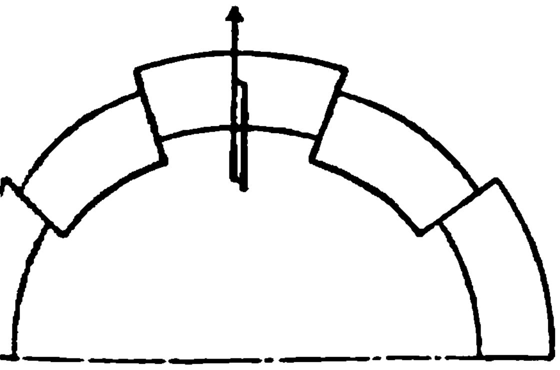


Fig. 7. Equilibrium of Forces on Voussoir



...8. Failure of Semicircular Arch. Haunches Sliding Down

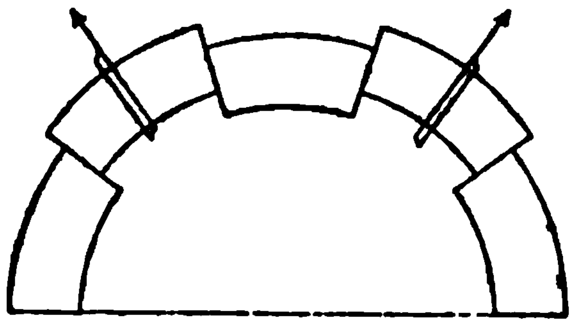


Fig. 9. Failure of Semicircular Arch. Haunches Sliding Up

...the CRITERIA OF SAFETY, explained in Chapter VII in the discussion of ...stability of a Buttress, will not be satisfied; and at any joint where these ...tions do not obtain, the voussoir above the joint will tend to SLIDE along ...joint-plane if the angle made by the thrust with a normal to the joint is ...r than the angle of friction. If the center of pressure lies outside the ...e third, there will be a tendency for the voussoir to OVERTURN. When ...tendencies reach extreme limits actual FAILURE may occur. Figures 8, 9, ...d 11 illustrate some of the ways in which an arch may fail, Figs. 8 and 9,

...is line is called, interchangeably, the LINE OF PRESSURE, the LINE OF RESISTANCE, ...RESISTANCE-LINE, etc. (See, also, Chapter XXXI, pages 1225 and 1234.)

showing different parts of the masonry sliding on the joints and Figs. 10
11 the failures caused by the passing of the line of pressure near the intrad
extrados.

Before passing to the actual discussion of the GRAPHIC METHOD for determ
the stability of arches, a consideration of the action of the STRESSES deve
in a construction of this kind will assist in a clearer understanding of the su

Fig. 8 shows how, if the line of resistance along the HAUNCHES of the
should turn sharply downward and in so doing make with a normal to o
the joints an angle greater than the angle of friction, the voussoirs at this

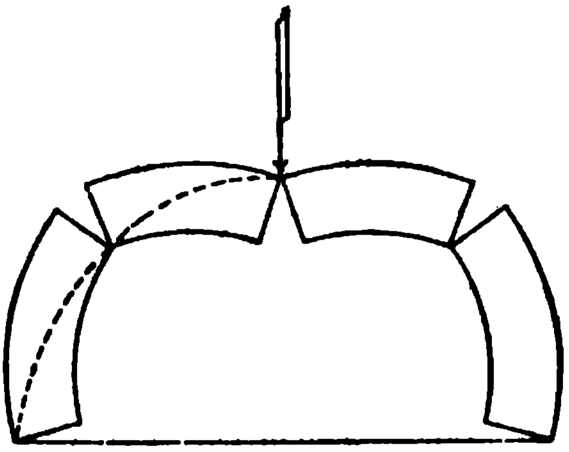


Fig. 10. Failure of Semicircular Arch. Opening of Arch-ring

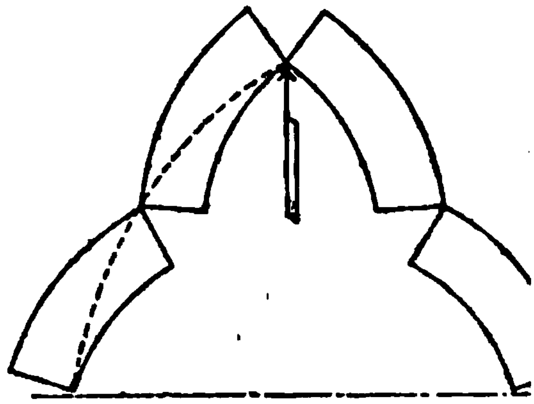


Fig. 11. Failure of Pointed A Opening of Arch-ring

would tend to slide inward on their joint-planes, forcing outward the vo
at the spring and crown of the arch. Fig. 9 shows how failure of the arch
occur under similar conditions, but with the line of resistance turning s
upward instead of downward. In these two cases it is conceivable th
though the RESISTANT THRUST at the joint where failure takes place mal
angle with the normal greater than the angle of friction, its point of appli
is still within the middle third of the joint.

Figs. 10 and 11, on the contrary, illustrate methods of failure in whi
though the angle made by the thrust may be such as to cause no SLIPPING
joint on another, its point of application is sufficiently outside the middl
of the arch-ring itself at the crown to cause OVERTURNING. In Fig. 10 t
of resistance passes high up, or perhaps entirely outside of the arch-ring,
voussoirs at the CROWN of the arch and low down along the HAUNCHE
Fig. 11 exactly contrary conditions exist.

The ten ways in which a masonry arch may fail have been classified as fol
"(1) By CRUSHING of the masonry; (2) By SLIDING of one voussoir upon a
(3) By one voussoir or section of masonry OVERTURNING about an a
voussoir or section; (4) By SHEARING in a horizontal or vertical plan
applying to solid concrete arches and not to voussoirs; (5) AS A COLUMN
the ratio of the unsupported length of an arch to its least width is great
twelve; (6) From STRIKING THE CENTERING before the mortar is hard o
the arch, although stable under the full load, is not stable under its
alone; (7) By STRIKING THE CENTERING or loading the arch during co
tion unsymmetrically; (8) By SETTLEMENT of the foundations; (9) By s
upon the foundations; (10) By OVERTURNING about any point in the
abutment. Methods (8) and (9) are the most common ways of failur
methods of failure, however, must be guarded against in design."

While some of these ways of failure may seem other than those illu
in the foregoing figures, they may be perhaps more properly considered

* W. J. Douglas in American Civil Engineering Pocket-Book, page 625.

...ure than ways of failure; and all, with the exception of the first, ...bout a position of the line of resistance in the arch-ring which causes ...in one of the ways noted.

...regard to the method of failure (1), the conditions may be such that the ...ing, although symmetrical, is so excessive that although the line of resistance ...ins within the middle third, the total pressure on a joint is sufficient to ...sh the material of which the arch is constructed. Such conditions, how-...r, are not common.

...m the foregoing discussion it is evident that in order to determine whether ...t a given arch is stable, it is necessary to find the true line of resistance ...sponding to the conditions of loading, form and dimensions of that par-...lar arch. It is always possible, in every arch-ring, to pass one maximum ...one minimum line of resistance. The true line of resistance will lie ...where between these two. The method of procedure, therefore, is to pass ...atively, a line of resistance, either a maximum or a minimum one, and see ...t remains within the middle third. If it does not, as it may not be the true ...of resistance, it does not mean necessarily that the arch is not stable. The ...t step then, is to note where it departs farthest from the middle third, and ...ss a second line of resistance through the same point on the crown-joint ...the point on the line of the middle third where the original line departs ...est from the middle third. If this second line of resistance remains within ...middle third it is reasonable to assume that the arch is stable. In these ...ese operations it is only necessary to consider half the arch when the loading ...mmetrical, and this is usually the case in architectural problems. The ...er of voussoirs, also, into which we divide the half-arch, is immaterial ...the joints need not coincide with those of the actual arch.

...order to pass a line of resistance through an arch-ring, the thrust exerted ...the other half at the crown-joint on the half-arch is first determined. ...thrust is then combined with the resultant of the weight of the first voussoir ...its load to determine the thrust exerted by this voussoir on the one next ...it, and this thrust, in turn, is combined in the same way with the resultant ...e weight and the load of the second voussoir, and so on down to the spring-...oint, for each succeeding voussoir. The points in which the various lines ...enting the thrusts cut the joints are known as the centers of pressure, ...the line joining them is the line of pressure or line of resistance. In ...rming this operation, the center of gravity of each voussoir as well as ...line passing through the center of gravity of the whole half-arch must be ...nd. The face of each voussoir may be considered a trapezoid, and any ...of the methods for finding the center of gravity of this figure may be used ...nding the center of gravity of each voussoir. The method of dividing the ...zoid into triangles is here employed and is shown at the side of the arch ...ig. 12. (See, also, in Chapters VI and VII.) As the determination ...e position of the line passing through the center of gravity of the half-arch ...e problem of finding the resultant of a system of parallel forces, the ...od involving the drawing of the equilibrium-polygon may be used. ...most convenient way to determine the stability of an arch is to use the ...ic method. The steps in this method are outlined in the preceding para-...h. Each of the operations will now be considered in detail.

...t Step. Draw one-half the arch to as large a scale as convenient, and ...e it into voussoirs of equal size. In the example shown in Fig. 12, the ...ing is divided into ten voussoirs of equal face-areas. As already pointed ...it is not necessary that these should represent the actual voussoirs of which ...ch is built. Next, the face-area of each of these voussoirs is to be found.

Where the arch-ring is divided into voussoirs of equal size, this is most
done by computing the total area of the arch-ring and dividing this total
by the number of voussoirs. The FORMULA for finding the area of one-ha
arch-ring is as follows:

$$\text{Area in square feet} = 0.7854\,(r^2 - r_1^2)$$

In this formula r is the outside radius and r_1 the inside radius in feet.
In this problem, for example, if the

$$\text{Area of the arch-ring} = 0\ 7854\,(12.5^2 - 10^2) = 44.2 \text{ sq ft}$$

as there are ten equal voussoirs, the area of each voussoir is 4.42 sq ft.
ing drawn out one-half of the arch-ring, divide the crown-joint into three parts, and with of $O'E$ and $O'F$ de the arcs dividin arch-ring into thi

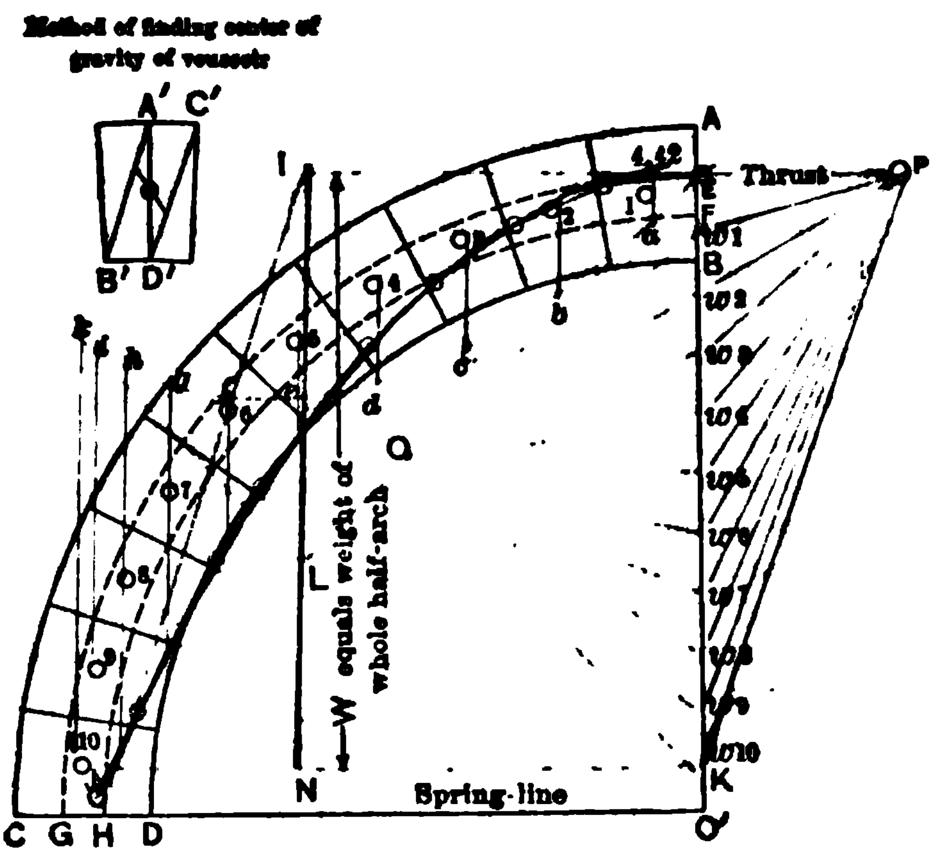


Fig. 12. Line of Pressure in Unloaded Semicircular Arch-ring

Second Step. C the points E a through which t a MINIMUM LINE SISTANCE. The F and G, through a MAXIMUM LINE SISTANCE can be could equally we been chosen. It be noted that a loaded semicir arch is more apt by opening at t trados at the and at the extra the haunch, and fore, in this cas line of resistance ably passes nearer the OUTER THIRD at the CROWN and nearer the INNER at the HAUNCH. To determine this MINIMUM LINE OF RESISTANCE the MI THRUST, applied at the point E of the crown-joint, must first be determi

The half-arch is in equilibrium under the action of three forces: (THRUST AT THE CROWN, acting horizontally, applied at the point E and prev the half-arch from overturning inward; (2) the WEIGHT OF THE HAL considered as a vertical force, acting through its center of gravity and t to overturn it inwards about the point D; and (3) A FORCE EQUAL AND SITE TO THE RESULTANT of these two forces and passing from H to I. I intersection of the weight-line through the center of gravity of the hal with the line of action of the thrust at the crown, prolonged. It is thus p to construct the TRIANGLE OF THESE THREE FORCES and determine the magn of the thrusts, when the position of the weight-line of the half-arch is mined. It is first necessary to draw a vertical line through the center of g of each voussoir. The center of gravity of one of the voussoirs may be by the METHOD OF TRIANGLES, as shown in the supplementary figure at t of the arch-ring.

Having determined the positions of the centers of gravity of the vo

them on the voussoirs as shown. From the point E (Fig. 12) lay off vertically, to a scale of so many SQUARE UNITS TO A LINEAR UNIT, the area of each voussoir, one below the other, commencing with the top voussoir. The length of the line EK will then equal the total area of the arch-ring. From E and (Fig. 12) draw 45° lines intersecting at O. Draw Ow 1, Ow 2, Ow 3, etc. Then where OE intersects the first vertical line through the center of gravity of the first voussoir at a, draw a line parallel to Ow 1, intersecting the second vertical at b. Draw bc parallel to Ow 2, cd parallel to Ow 3 and so on to k. Draw kL parallel to Ow 10 and prolong it downward until it intersects EO prolonged, at L. A vertical line drawn through L will pass through the center of gravity of the half arch-ring. This is an application to a practical problem of the method of finding, by the EQUILIBRIUM-POLYGON, the line of action of the resultant of a SYSTEM OF PARALLEL FORCES. The weights of the individual voussoirs act along parallel vertical lines and the weight of the half-arch is their resultant in magnitude.

Third Step. To determine the THRUST AT THE CROWN and the REACTION AT THE SPRING, draw a horizontal line through E, the upper part of the middle third, and a vertical line through L, the two lines intersecting at I (Fig. 12). For the arch to be stable, it is, in general, considered necessary for the LINE OF RESISTANCE to pass within the MIDDLE THIRD. First, assume that the line of pressure or resistance starts at E and comes out at H. Draw a line IH the direction of the line of action of the resultant of the thrust at the crown and the weight of the half-arch, and draw, also, a horizontal line opposite the point w 10, between N and M. This horizontal line MN represents the magnitude of the horizontal thrust at the crown, for INM is the TRIANGLE OF THE THREE FORCES in equilibrium, the THRUST at the crown, the WEIGHT of the half-arch and the REACTION at the spring. Draw w 10 O^p parallel to HI, and the lines O^pw 1, O^pw 2, O^pw 3, etc. O^pE, equal to NM, is the thrust at the crown, and w 10 O^p, equal to MI, the reaction at the spring. INM and EKO^p are similar triangles.

Fourth Step. It is required next, to determine the LINE OF RESISTANCE through the arch-ring. The thrust at E is combined with the weight of the first voussoir; their resultant is found and in turn combined with the weight of the second voussoir; and so on for all the voussoirs. The intersections of these resultants with the joint-lines are the CENTERS OF PRESSURE; the line joining the centers of pressure is the LINE OF RESISTANCE.

These resultants could be determined by drawing a series of PARALLELOGRAMS OF FORCES over each voussoir. This would complicate the figure and involve unnecessary labor. It is found more convenient to draw the TRIANGLES OF FORCES one after the other, at the right-hand side of the figure and then transfer the results thus obtained by means of parallel lines to the figure itself, especially as the weights of the voussoirs have already been laid off along the line EK, at Ew 1, w 2, w 3, w 4, w 5, etc.

Then from the point where O^pE prolonged intersects the first vertical in voussoir number 1, draw a (green) line to the second vertical, parallel to O^pw_1; from this point, a (green) line to the third vertical, parallel to O^pw 2 and so on. The last line should pass through H. Join the various points, where these (green) lines cut the joints at the centers of pressure, by the broken (red) line. The last line drawn is the LINE OF RESISTANCE. If this line lies entirely within the MIDDLE THIRD of the arch-ring, the arch may be considered to be stable. Suppose that the line of resistance passes not only outside of the middle third but also outside of the arch-ring itself; it is still possible that the arch is unstable. This is the case in Fig. 12 and we will next determine if a

line of resistance can be drawn which will remain within the limits of the mid third of the arch-ring.

Fifth Step. The Second Trial. Reproducing the condition of Fig. 12 Fig. 13, without the construction lines, it is seen that the LINE OF RESISTA leaves the arch-ring at *R* enters it again at *S*, while furthest from it at *U*. If, at a perpendicular is erected straight line joining the points *R* and *S*, this per dicular line *VW*, called the I OF FRACTURE, will be appr mately the trace of the p along which, with the lin resistance under considera the arch will tend to fail, sumedly by TURNING OVE the right about the poin This shows that the THRUS THE CROWN, assumed to applied at the point *E*, wh sufficient intensity to main equilibrium about *H*, is n sufficient intensity to mai equilibrium about *V*. If a SECOND THRUST, of suff intensity to maintain equilibrium about *V*, or better, about *X*, can be ap at *E* without being so great in magnitude that it will OVERTURN THE OUTWARD about *G*, or some other point on the outer line of the middle thi

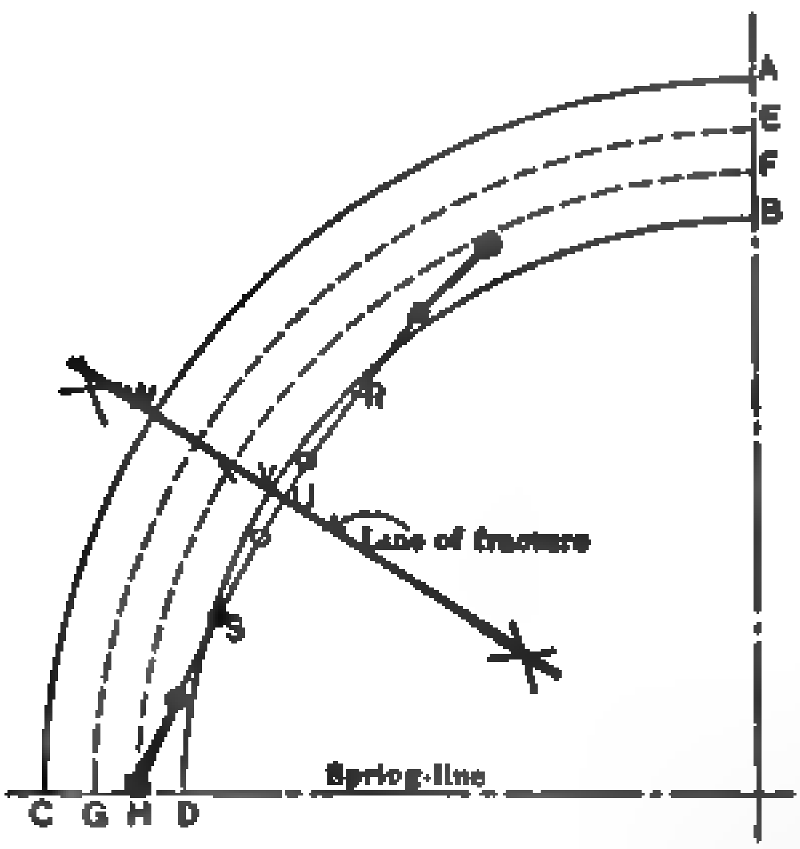


Fig. 13. Line of Fracture in Unloaded Semicircular Arch-ring

Fig. 14. Second Line of Pressure in Unloaded Semicircular Arch-ring

is reasonable to conclude that the line of resistance resulting from this t very nearly the TRUE LINE OF RESISTANCE in the arch-ring and that t is stable.

In order to determine this NEW LINE OF RESISTANCE the NEW THRUST

… must be found (Fig. 14). The preliminary steps required for this are … same as before until the seventh voussoir is reached. This is divided into … voussoirs by the line VW (Fig. 14), one being $w6\ w6^a$ and the other the … mainder of this seventh voussoir, and this division must be allowed for along … load-line EK, at $w6\ w6^a$. The line $w6\ w6^a$ represents the area of vous… 6^a, and the line $w6^a\ w7$ the area of the remainder of the seventh voussoir. The vertical line IL, passing through the center of gravity of that part of the … arch above the line VW, is found by prolonging backwards the line hg, … lel to $O\ w6^a$, until it intersects OE at L. To find the NEW THRUST AT THE … by completing the TRIANGLE OF FORCES for this thrust and the force … and opposite to their resultant, the inclined (blue) line must be drawn … gh the point X and the horizontal (blue) line through $w6^a$. The new thrust … is as before NM, equal to O^pE. This thrust is laid off at O^pE, the (green) … $O^pw\ 1$, $O^pw\ 2$, $O^pw\ 3$, etc., being drawn as before and the new line of re… … being drawn through the points where the parallels to these (green)

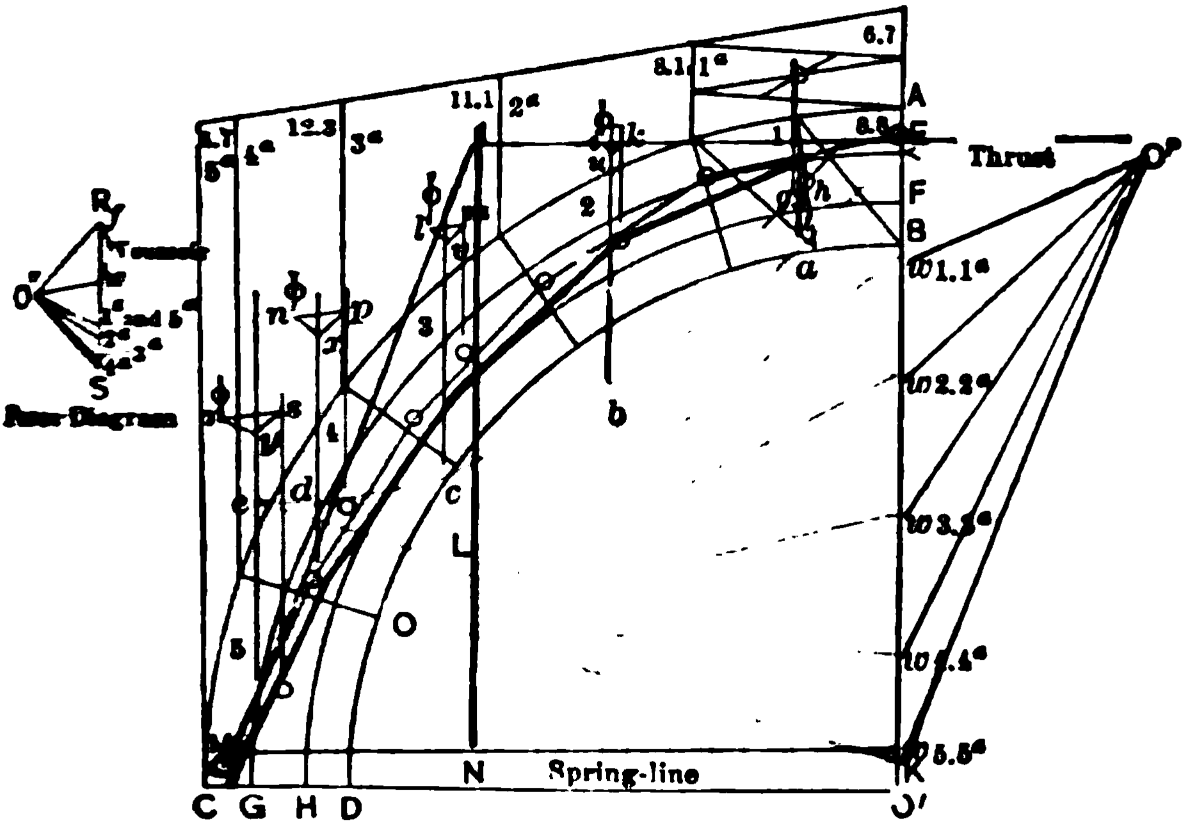


Fig. 15. Line of Pressure in Loaded Semicircular Arch-ring

… cut the joints. This NEW LINE OF RESISTANCE, if drawn correctly, should … through X. It lies within the middle third, except for a short distance at … springing, and hence it is justifiable to consider the arch stable. If it had … outside the middle third to any great extent, in this second trial, this … mption would not have been justified.

… is discussion explains the method of determining the stability of an UN… … SEMICIRCULAR ARCH. Such cases very seldom occur in practice, but … serve to illustrate the methods which apply generally to all other cases. … LOADED ARCH-RINGS there is slight difference in the method of determining … position of the center of gravity.

… mple 3. A LOADED OR SURCHARGED SEMICIRCULAR ARCH (Fig. 15) will be … dered next. Assume the same arch shown in Figs. 12, 13 and 14, and sup… … to be loaded with a wall of masonry of the same thickness and weight … are foot as that of the arch-ring, the upper surface of the wall being an … plane, 1 ft above the arch-ring at the crown, and 8 ft above it at the … The assumption of the particular load in this case is a purely arbitrary

one for the purpose of illustrating the method of solution. The determin of the ACTUAL LOAD that comes upon an arch in any given case is by no r easy, so numerous are the uncertain elements that affect the transmiss this load to the arch-ring.

The customary procedure is to assume that the load is itself transmitted arch-ring VERTICALLY DOWNWARD. Each voussoir thus receives that porti the load which is included between two vertical lines drawn to the poi intersection of the joints on either side of that voussoir with the extrados. ing made this assumption it is necessary next to determine how much total superimposed masonry bears upon the arch-ring.

It is a matter of common observation that if an opening is made in a especially in a wall that has stood for some time, the major portion of the ma above this opening is self-supporting, limited portions only, bounded by a what irregular line, falling down into the opening, as shown in Fig. 16.

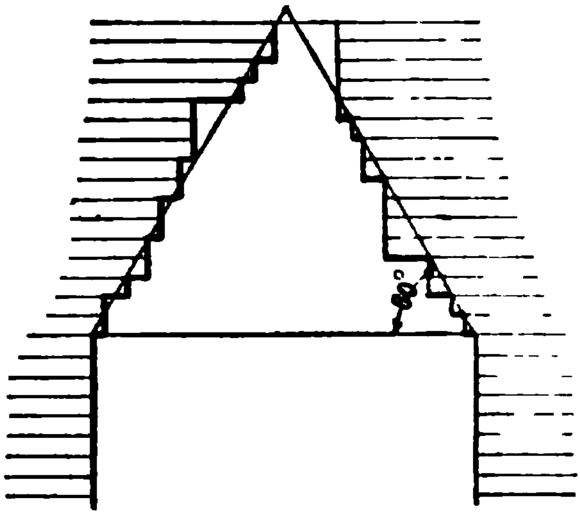


Fig. 16. Triangle of Loading over Opening

profile of this boundary-line depends the nature of the material of whic wall is constructed, the size of the s bricks, etc., the character of the and the quality of the mortar. being the case, all the masonry abo arch should not be considered a load on it. Some authorities recom considering as the proper load, for l work, a TRIANGULAR PART of the the sides of which triangle have a clination to the horizontal of 45°; assume an inclination of 60° (Fig. (See, also, Chapter XV, page 612.) exact determination of this loa mechanical laws is difficult if not impossible. It is better to consider case separately and by a careful study of the conditions to determine as c as possible just what portion of the weight of the superimposed maso transmitted to the arch. Having assumed a load for this particular arc (Fig. 15), the procedure is as follows:

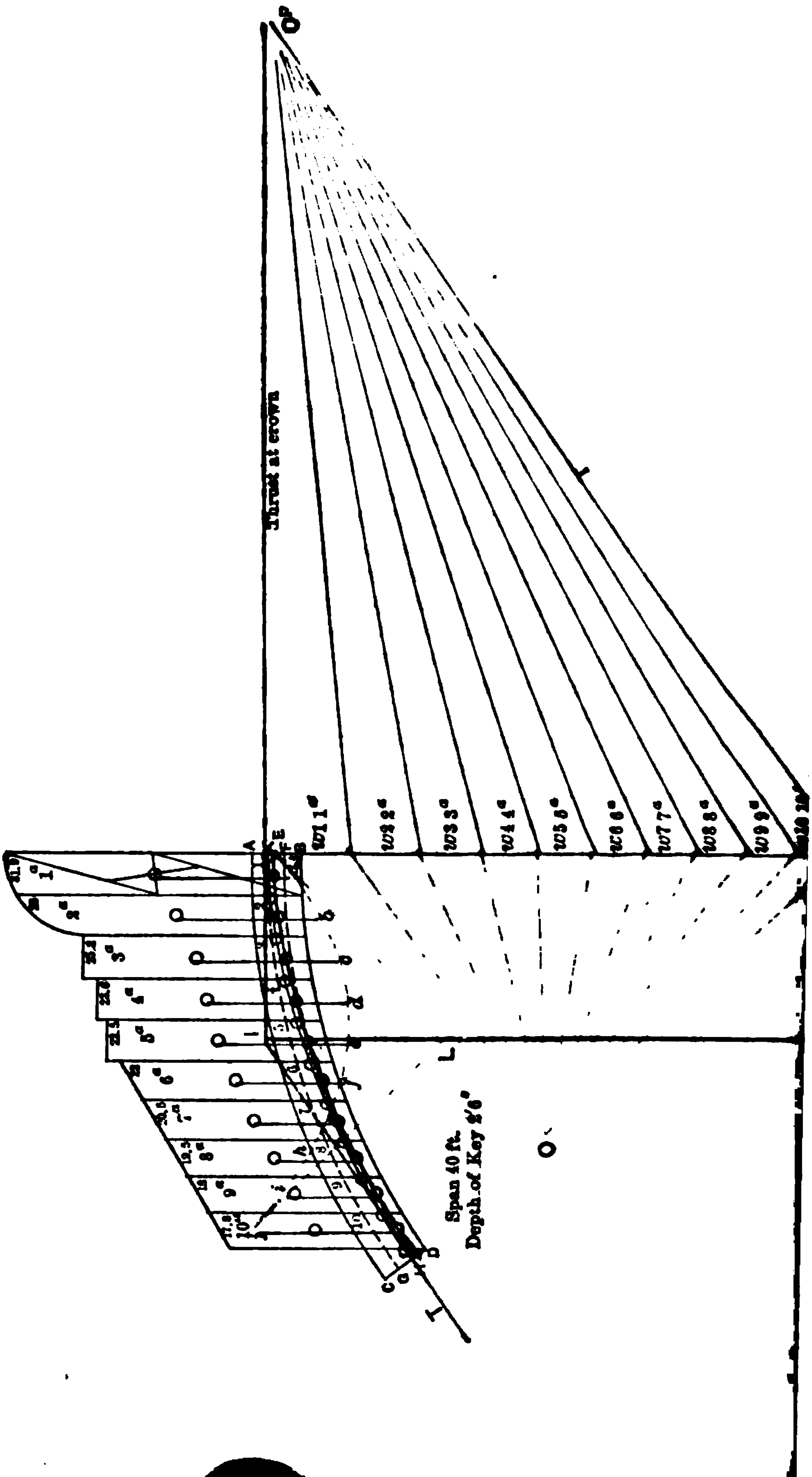
First Step of Example 3. This involves the finding of the CENTER OF GR of the ARCH-RING AND LOAD COMBINED. Divide the arch-ring into five vou of equal size. In this case the area of each voussoir is equal to 44.2 sq ft ÷ 8.8 sq ft. (See under First Step, Fig. 12, preceding example.) The surc or load, also, is divided into five parts, not necessarily equal, by drawin tical lines to the points of intersection of the joints and the extrados. Th proximate area of each one of these surcharges is found by multiplyin the sum of the lengths of the two parallel vertical sides by the length horizontal distance between them.

The positions of the center of gravity of each voussoir and of the cen gravity of each voussoir-surcharge are determined as in the preceding exa The CENTERS OF GRAVITY of these SURCHARGES can be found by dividing TRAPEZOIDAL FIGURE into TRIANGLES as shown, remembering that the M LINE in this case joins the middle points of the two parallel faces. As the are vertical, the medial lines approach a horizontal direction. This con tion is shown on surcharge 1^a, Fig. 15. Having drawn the lines of act the weights of the various voussoirs and of their loads through their resp centers of gravity, the lines of action of the combined weight of each vo and its load must be found. The construction for this operation is sho

left of Fig. 15. The method used, that of the EQUILIBRIUM-POLYGON, the same as that employed in the previous example to find the line passing through the center of gravity of the half-arch, only in this case the forces are reduced to two. Furthermore, as the areas of the various voussoirs are equal it is possible to superimpose the different FORCE-DIAGRAMS, one over the other, and so save considerable labor. Begin, therefore, by laying off along the line at the left of the loaded arch, and at any convenient scale, fw, the area (weight) of a voussoir; then from w, in turn, the distances $w\ 1^a$, $w\ 2^a$, $w\ 3^a$, etc., representing the areas of the successive surcharges, 1^a, 2^a, 3^a, etc., always at the same scale. The scale to be employed later for laying off the combined weights of the voussoirs and their loads along the line AK is the best one to choose, but a difference in scales is not important. In this particular instance the two points 1^a and 5^a coincide because the two areas 1^a and 5^a, although of different shapes, are each equal to 6.7 sq ft. This is a mere coincidence. Next draw f and $4^a\ O''$ at 45° to RS, and in turn, $O''w$, $O''1^a$, $O''2^a$, etc. As the problem which presents itself is to combine the weight of each voussoir with its individual surcharge, and as the weights of all the voussoirs are equal, and, furthermore, the forces which are to be combined to find their resultant are only two, the POLE-LINES or RAYS $O''f$ and $O''w$ in the FORCE-DIAGRAM serve in each case, and the FUNICULAR POLYGON is reduced to a TRIANGLE. Draw gh, ik, lm, np and rn parallel to $O''w$, and ht, ku, mv, px and sy parallel to $O''f$; and draw gt, iu, lv, nx and ry parallel respectively to $O''1^a$, $O''2^a$, $O''3^a$, $O''4^a$ and $O''5^a$. The points t, u, v, x and y are the points through which to draw the heavy (red) lines of action of the combined weights of the voussoirs and their surcharges. Having found and drawn these lines, the procedure for finding the line IN is the same as in the previous example, except that the distances $Ew1\ 1^a$, $w1\ 1^a$, 2^a, etc., instead of being equal to the weights of the voussoirs alone, are equal to the combined weights of each voussoir and its surcharge, $Ew1\ 1^a$, being equal to $f\ 1^a$, $w1\ 1^a$ to $w2\ 2^a$ being equal to $f\ 2^a$, etc.

The line EO is drawn at 45° to AO', but as the position of the POLE-POINT, is entirely arbitrary, the line $Ow\ 5\ 5^a$ has been drawn in this case in such a way that O falls well over toward the left of the figure, thus avoiding a certain amount of confusion in the drawing which would have resulted if $Ow\ 5\ 5^a$ made an angle of 45° with AO'. The lines ab, bc, cd and de are drawn respectively parallel to $w1\ 1^aO$, $w2\ 2^aO$, etc., and eL is produced backward parallel to $5\ 5^a$ until it intersects EO at L, which is the point through which the heavy (red) line IN, passing through the center of gravity of the whole half-arch and its surcharge, should be drawn. A vertical line drawn through L will pass through the center of gravity of the arch-ring and its load. If this were an arch used for a building and if the only abutments possible were of such size and shape that it was essential for the thrust exerted by the last or fifth voussoir on the abutments to approach more nearly the vertical, the architectural expedient of increasing slightly the weight of the surcharge, 5^a, on this voussoir by adding a piece of ornament, such as a cartouche, could be resorted to. A case of this kind in actual practice is the archway over the entrance to the service-court of the Grand Opera House in Paris, where the pyramidal stone ornaments that surmount the cornice on either side of the central motive were added after the original design was made, with this end in view. In the example illustrated in Fig. 15 the areas of the faces of the surcharges are shown by the figures on the faces. For the second surcharge from the crown, for example, the area is ... sq ft.

Second Step of Example 3. This involves the determination of the THRUST at the CROWN and the LINE OF RESISTANCE. The method of finding this thrust


Thrust at crown
Span 40 ft.
Depth of Key 2'6"

the crown is similar to that employed in the previous example. In that example, however, it was found that this thrust, applied at E and determined assuming H as the point of application of the reaction at the spring, produced a line of resistance which fell considerably below the middle third. But instead of repeating the operations required by a second trial, as in the previous example, the expedient is tried of slightly increasing the inclination to the vertical of the (blue) line IM, and so assuming a somewhat greater THRUST AT THE CROWN. As the line of resistance, as shown in Fig. 15, passed with this thrust departs but slightly from the middle third near the springing, we are justified in assuming that this arch is stable under the given conditions. The method used for this example may be used, also, for a SEMIELLIPTICAL ARCH.

Example 4. This example (Fig. 17) illustrates the application of the preceding methods, with some variations, to the determination of the position of the center of gravity of a LOADED SEGMENTAL ARCH, the thrusts at the crown and springing and the line of pressure or resistance through the arch-ring. In this case, instead of dividing the arch-ring into a certain number of voussoirs with joints radiating from a center and considering the surcharge on each individual voussoir, the method of dividing the arch-ring and its load into VERTICAL SLICES, in this case two feet wide, and computing the areas of the entire slices has been adopted. Having computed the areas of the slices, including in each case the combined area of the sliced part of the arch-ring and its surcharge, we lay them off in order on E, to a convenient scale, and then proceed as in the previous examples. The remaining steps required to determine the thrusts at the crown and at the springing and the line of resistance are also the same as explained in the foregoing paragraphs. In a FLAT SEGMENTAL ARCH there is practically no need of dividing the arch-ring into voussoirs by JOINTS RADIATING FROM A CENTER, in order to determine its stability. Of course, when built, they must be made to radiate. Fig. 17 shows the GRAPHICAL ANALYSIS of an arch of 40-ft span and carrying a load $13\frac{1}{2}$ ft high at the crown. The depth of the arch-ring is 2 ft 6 in. It is seen that the line of resistance lies entirely within the middle third, and that the arch is therefore stable. It is to be noted that the line of resistance in a SEGMENTAL ARCH should be drawn through the LOWER or INNER EDGE of the middle third at the springing. It is to be noted, also, that the horizontal thrust at the crown and the thrust T against the supports are very great when compared with those in a SEMICIRCULAR ARCH; and hence, although the SEGMENTAL arch is the stronger of the two, it requires much heavier abutments. The foregoing examples serve to show the various methods of determining the stability and thrusts of any arch used in buildings.

CHAPTER IX

REACTIONS AND BENDING MOMENTS FOR BE

By

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1. Reactions for Simple Beams

Definition of Reaction. One of the fundamental principles of static rium is that the sum of all the forces acting upon a body in one direction balanced by the sum of another set of forces acting in the opposite d Therefore, in the case of a beam or girder, the loads acting downward balanced by an equal set of forces at the supports, acting upward. T ward forces are called THRUSTS, or REACTIONS and in computing the of beams one of the first steps is to determine them, since the loads are given in intensity and position.

The Principle of Moments. The reactions may be determined application of another fundamental principle of static equilibrium fo acting in the same plane. The algebraic sum of the moments of all th

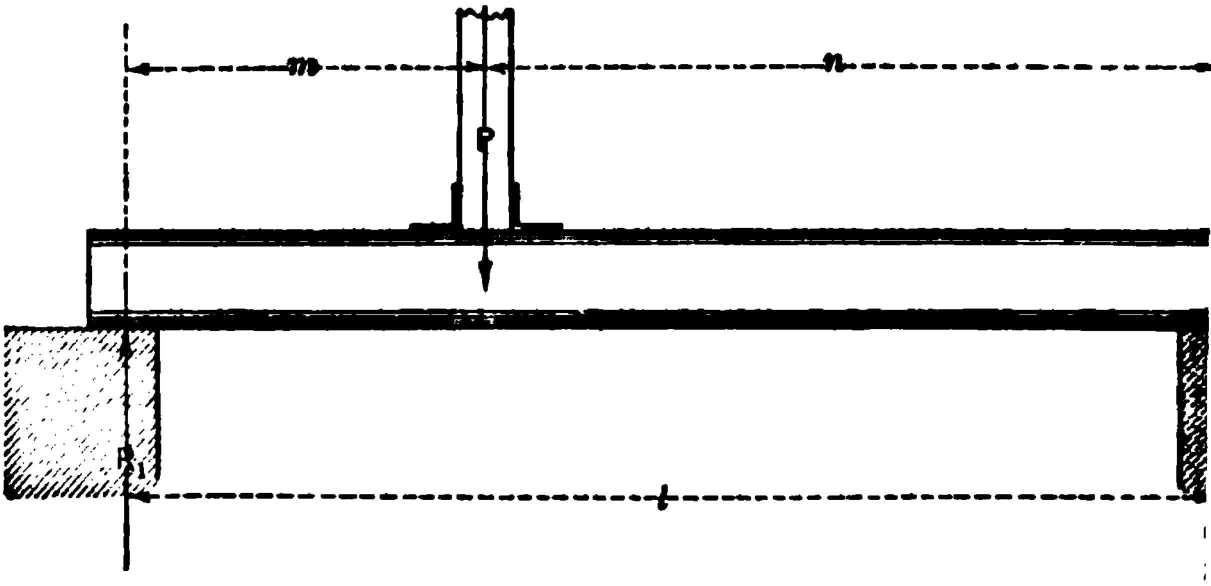

Fig. 1. Simple Beam. One Concentrated Load

taken about any point in the plane in which they act must be zero. The OF A FORCE about a point is the product of the magnitude or intensity of by the perpendicular distance between the LINE OF ACTION of the force point. The perpendicular distance is called the LEVER-ARM, and the CENTER OF MOMENTS. Forces acting upward are considered POS those acting downward are considered NEGATIVE. The center of mom be taken at any point in the plane of action of the forces, but it convenient to take it at one of the reactions. For example, the beam supports a concentrated load P at the distance m from the left support. the left reaction take the center of moments at the right reaction. EQUATION OF MOMENTS is

$$R_1l - Pn = 0.$$

from which $$R_1 = Pn/l$$

In like manner, to find R_2 the center of moments is taken at R_1 and the equation of moments is

$$R_2l - Pm = 0, \text{ from which } R_2 = Pm/l \qquad (1)'$$

From the first principle of statics mentioned, $R_1 + R_2$ must equal P; hence, as a check, $(Pn/l) + (Pm/l) = P$.

Example 1. Let a beam 15 ft in span support a concentrated load of 700 lb, 6 ft from the left end; or, $P = 700$, $m = 6$ and $n = 9$. Then, from Formula (1). $R_1 = (700 \times 9)/15 = 420$ lb. $R_2 = (700 \times 6)/15 = 280$ lb, and $420 + 280 = 700$ lb. For a concentrated load at the middle, or for a uniform load over a simple beam, it is evident without applying the conditions of equilibrium, that each reaction is one-half the load, for, in Formulas (1) and (1)′, m and n each equal $\frac{1}{2}l$ and R_1 and $R_2 = \frac{1}{2} P$.

For any number of concentrated loads (Fig. 2) the reactions may be found by adding together the reactions found by Formula (1) due to each load separately, or they may be computed in one operation by the following formula:

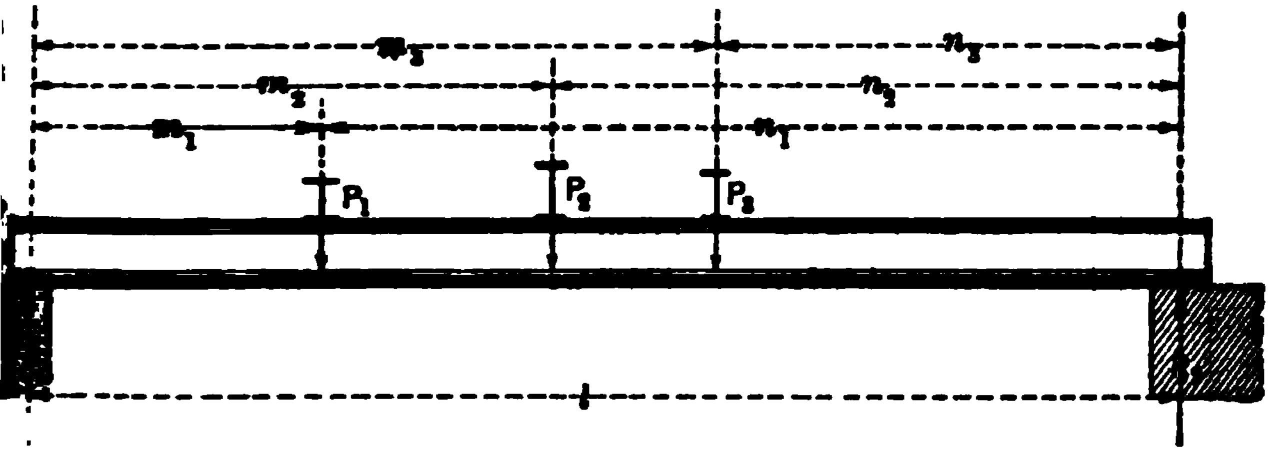

Fig. 2. Simple Beam. Three Concentrated Loads

To find the right reaction, the center of moments is taken at the left support, and the equation of moments is

$$R_2l - P_1m_1 - P_2m_2 - P_3m_3 = 0$$

hence,

$$R_2 = \frac{P_1m_1 + P_2m_2 + P_3m_3}{l} \qquad (2)$$

In like manner, to find R_1 the center of moments is taken at R_2 and the equation of moments is

$$R_1l - P_1n_1 - P_2n_2 - P_3n_3 = 0$$

from which

$$R_1 = \frac{P_1n_1 + P_2n_2 + P_3n_3}{l} \qquad (3)$$

Example 2. Suppose the beam in Fig. 2 is 20 ft in length. Let there be three concentrated loads of 500, 800 and 600 lb placed 5, 9 and 12 ft respectively from the left support. Then $l = 20$, $m_1 = 5$, $m_2 = 9$, $m_3 = 12$, $P_1 = 500$, $P_2 = 800$ and $P_3 = 600$. Substituting in Formulas (2) and (3),

$$R_2 = \frac{500 \times 5 + 800 \times 9 + 600 \times 12}{20} = 845 \text{ lb}$$

$$R_1 = \frac{500 \times 15 + 800 \times 11 + 600 \times 8}{20} = 1\,055 \text{ lb}$$

$$500 + 800 + 600 = 845 + 1\,055 = 1\,900 \text{ lb}$$

To find the reactions for a combination of uniformly distributed and centrated loads, to each of the reactions obtained by Formulas (1) or (2) f concentrated loads, add one-half the distributed load. Thus, suppos 20-ft beam in this example weighs 40 lb per linear ft. This is considere uniformly distributed load and for the entire beam it is 40 lb × 20 = 8 By the rule, one-half of this is added to each reaction, so that the total rea are, $R_2 = 845 + 400 = 1\,245$ lb and $R_1 = 1\,055 + 400 = 1\,455$ lb.

Example 3. For a distributed load applied over only a part of the span, Fig. 3, assume the load to be CONCENTRATED AT THE MIDDLE of the part

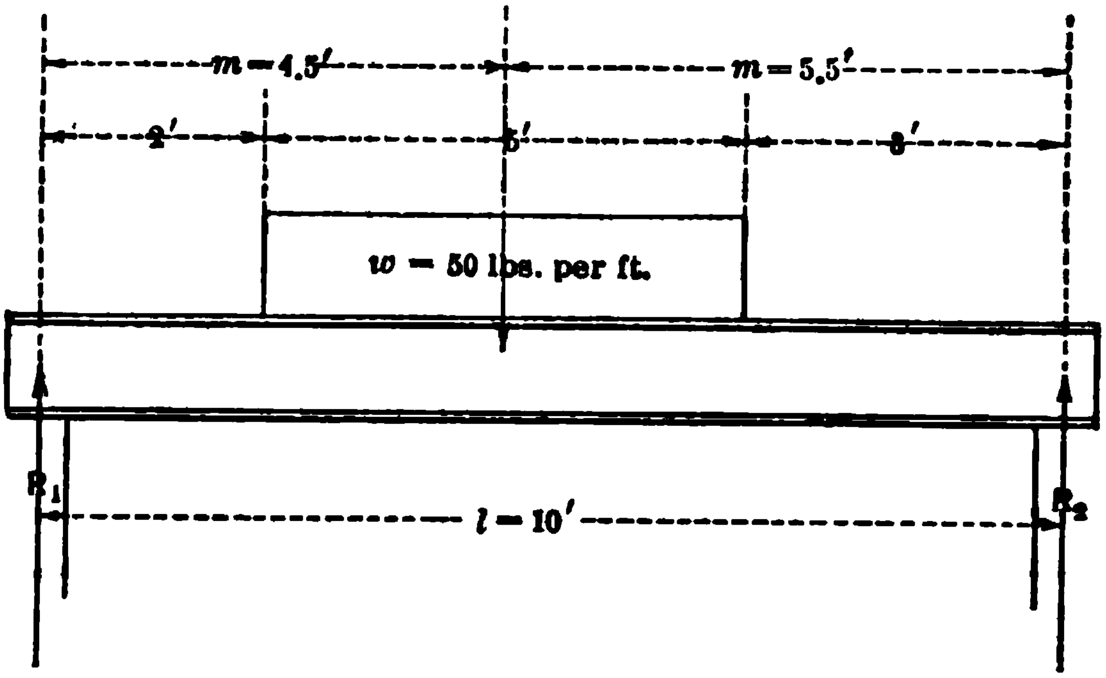

Fig. 3. Simple Beam. Distributed Load over Part of Span

which it acts and use Formulas (1) and (1)'. For example, let w (Fig. 3) eq lb per linear ft, applied for a distance of 5 ft over the beam. Then W, the load, is 50 lb × 5 = 250 lb. This may be assumed to be concentrated center, 4.5 ft from the left support. Then $P = 250$, $m = 4.5$ and $n = 5.5$ from Formulas (1) and (1)',

$$R_1 = \frac{250 \times 5.5}{10} = 137.5 \text{ lb}$$

and

$$R_2 = \frac{250 \times 4.5}{10} = 112.5 \text{ lb}$$

Therefore, for any combination of concentrated and uniform loads distri over the entire beam, or over only part of it, find the reactions due to the centrated loads by Formulas (1) or (2), and to them add the reactions due t uniformly distributed loads.

2. Bending Moments in Cantilever and Simple Beams*

Definitions. The bending moment is a measure of the tendencies of for break a beam by BENDING OR FLEXURE. Fig. 4 shows the manner in wh simple beam, supported at the ends, breaks when subjected to a load gr than it can bear. The effect of a load upon a beam is to cause it to SA BEND. The bending of the beam shortens, or compresses, the upper fiber stretches, or elongates, the lower fibers. So long as the resistance of the

* See, also, Chapter XV, pages 555 to 564.

shortening, or compression, and to stretching, or tension, is greater than the tendency of the load to disrupt them, the beam carries the load; but, when the load causes a greater tension, or compression, on the fibers than they are capable of resisting, the beam breaks. The stretching of the fibers before breaking allows the beam to bend; hence, the name BENDING MOMENT has been given to the forces causing a beam to BEND and perhaps ultimately to BREAK.

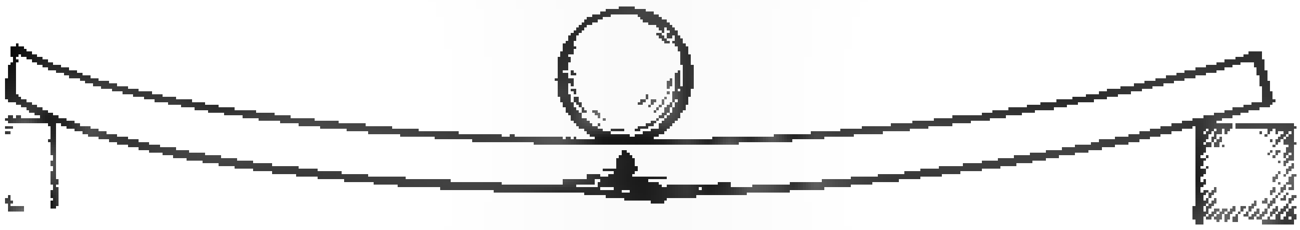

Fig. 4. Manner of Rupture of Simple Beam

In order to calculate the FLEXURAL STRENGTH OF A BEAM, it is necessary to ascertain the nature and extent, first, of the EXTERNAL FORCES acting to break the beam, and secondly of the INTERNAL FORCES OF STRESSES tending to resist rupture.* The external forces tending to break the beam by flexure are the DOWNWARD LOADS and the UPWARD REACTIONS. Each acts with a LEVERAGE equal to the perpendicular distance from its LINE OF ACTION to the section at which the beam tends to break. The algebraic sum of the moments of these external forces on the left, or right, of any section is called the BENDING MOMENT at that section, since it is the MOMENT OF THE RESULTANT OF THE FORCES which tends to bend the beam at that section. It is generally designated by M. Then, from the definition, the BENDING MOMENT for any section of a beam resting on two supports and in a state of flexure under a load or loads is M = the moment of either reaction minus the sum of the moments of the loads between that reaction and the section. The moment of the reaction is UPWARD, or POSITIVE, and the moment of any load DOWNWARD, or NEGATIVE, if the part of the beam on the left of the section is considered.

Bending Moments in Beams for Different Kinds of Loading

Case I

Beam Fixed at One End and Loaded with a Concentrated Load P, Near the Free End (Fig. 5).

Fig. 5. Cantilever Beam. Concentrated Load near Free End

Maximum bending moment, at wall $= P \times l$

Bending moment at any other section $x = Px$

Note. If l is in feet, the bending moment will be in foot-pounds; if l is in inches, the bending moment will be in inch-pounds.

Case II

Beam Fixed at One End and Loaded with a Uniformly Distributed Load W (Fig. 6)

Maximum bending moment, at wall $= W \times l/2$

At any other section x, $M = wx \times x/2 = wx^2/2$

Note. $W = wl$ and w = the load per unit of length.

* See Chapter X for a discussion of these internal stresses and of the resisting moment.

Case III

Beam Fixed at One End and Loaded with Both a Concentrated and a Uni Distributed Load (*Fig. 7*).

Maximum bending moment, at wall = $P \times l_2 + W \times l_1/2$

Fig. 6. Cantilever Beam. Uniformly Distributed Load

Fig. 7. Cantilever Beam. Distr uted Load and Load at Free En

Case IV

Beam Supported at Both Ends and Loaded with a Concentrated Load Middle (Fig. 8).

Maximum bending moment, under the load = $Pl/4$

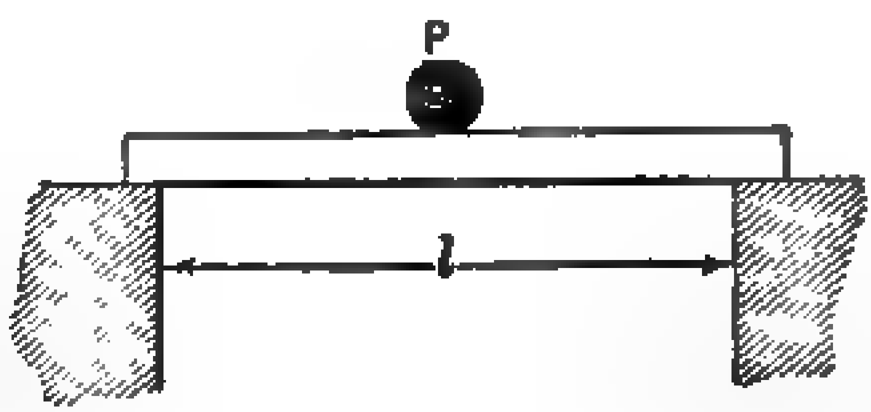

Fig. 8. Simple Beam. Concentrated Load at the Middle

Case V

Beam Supported at Both Ends and Loaded with a Uniformly Distr Load *W* (Fig. 9).

Maximum bending moment, at the middle = $Wl/8$

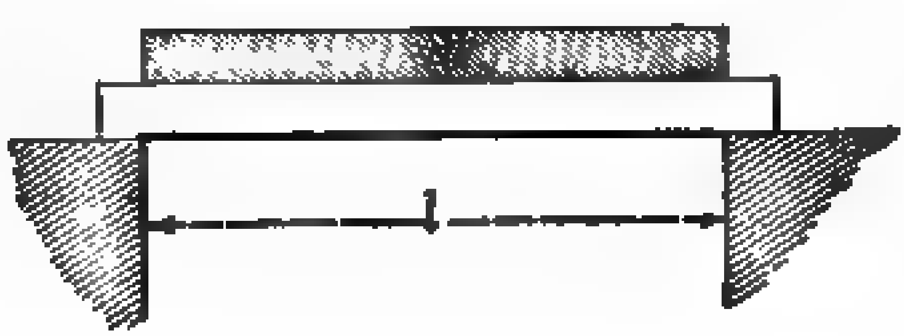

Fig. 9. Simple Beam. Uniformly Distributed Load

Case VI

m Supported at Both Ends and Loaded with a Concentrated Load not at the dle (Fig. 10).

ximum bending moment, under the load = Pmn/l

Fig. 10. Simple Beam. Concentrated Load not at the Middle

Case VII

m Supported at Both Ends and Loaded Symmetrically with Two Equal ntrated Loads (Fig. 11).

ximum bending moment = Pm and is the same for any section of the beam een the two loads.

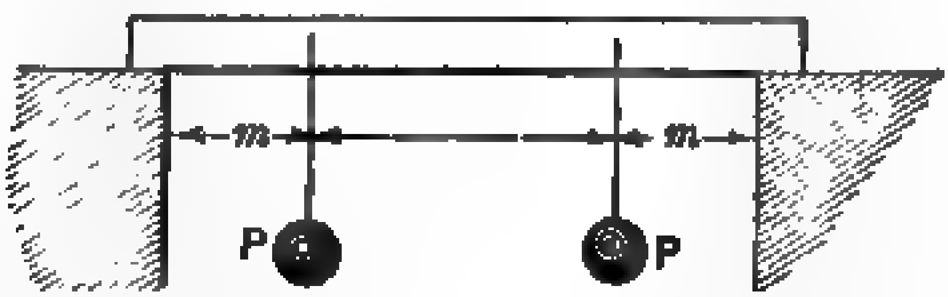

Fig. 11. Simple Beam. Two Concentrated Loads Symmetrically Placed

m these examples it will be seen that all the quantities which enter into computation of the bending moment are the load, the span and the distance e point of application of the load from the center of moments.

Case VIII

m Supported at Both Ends and Loaded with a Distributed Load Over Part e Span (Fig. 12).

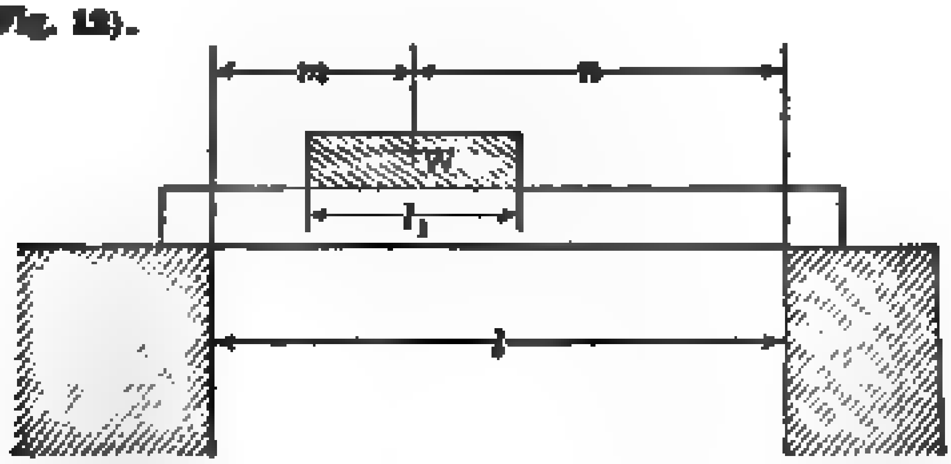

Fig. 12. Simple Beam. Distributed Load over Part of Span

med under the center of the load, $M^*_{max} = Wmn/l - Wl_1/8$
n m and n are equal the bending moment $= W \times l/4 - W \times l_1/8$

is is only approximately correct when m and n are unequal. For the exact value, e section of zero shear; the maximum bending moment will be at that section. lso, Example 5, page 561.)

Example 4. In Fig. 12 let $W = 800$ lb, $m = 8$ ft, $n = 12$ ft, $l = 20$ ft and l_1 =
Then the bending moment

$$= \frac{800 \times 8 \times 12}{20} - \frac{800 \times 8}{8} = 3\,840 - 800 = 3\,040 \text{ ft-lb}, \quad \text{or} \quad 36\,480$$

Example 5. In Fig. 12 let $m = n = 10$ ft, $l = 20$ ft. $l_1 = 4$ ft and $W = 6$
Then the bending moment

$$= \frac{600 \times 20}{4} - \frac{600 \times 4}{8} = 3\,000 - 300 = 2\,700 \text{ ft-lb}, \quad \text{or} \quad 32\,400 \text{ in-lb}$$

The Bending Moment for any Case Other Than the Above may easi obtained by the GRAPHIC METHOD, which will now be explained.

4. Graphic Method of Determining Bending Moments in Bea

Beam with One Concentrated Load (Fig. 13).

The BENDING MOMENT of a beam supported at both ends and loaded wi concentrated load may be determined GRAPHICALLY, as follows:

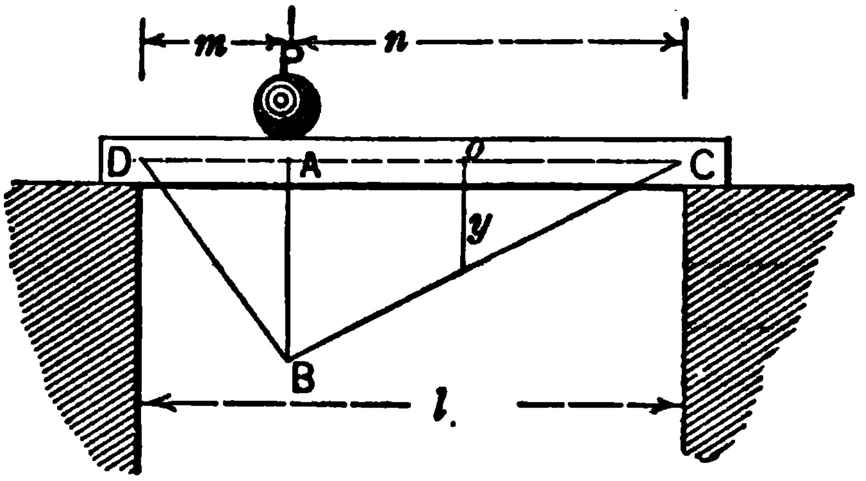

Fig. 13. Bending-moment Diagram. One Concentrated Load

Let P be the applied as shown. by the rule under Ca the MAXIMUM BE MOMENT is under th and $= Pmn/l$

Draw the beam, the given span, acc to scale, and measur the line AB, to a s FOOT-POUNDS to the INCH, a distance to the bending m Connect B with ea of the beam. To find the bending moment at any other point of the as at o, draw the vertical line y to BC. Its length, measured to the scale to which AB is drawn, will give the bending moment at o. The $DBCAD$ is called the BENDING-MOMENT DIAGRAM and the lines BD a are called INFLUENCE LINES for the bending moments.

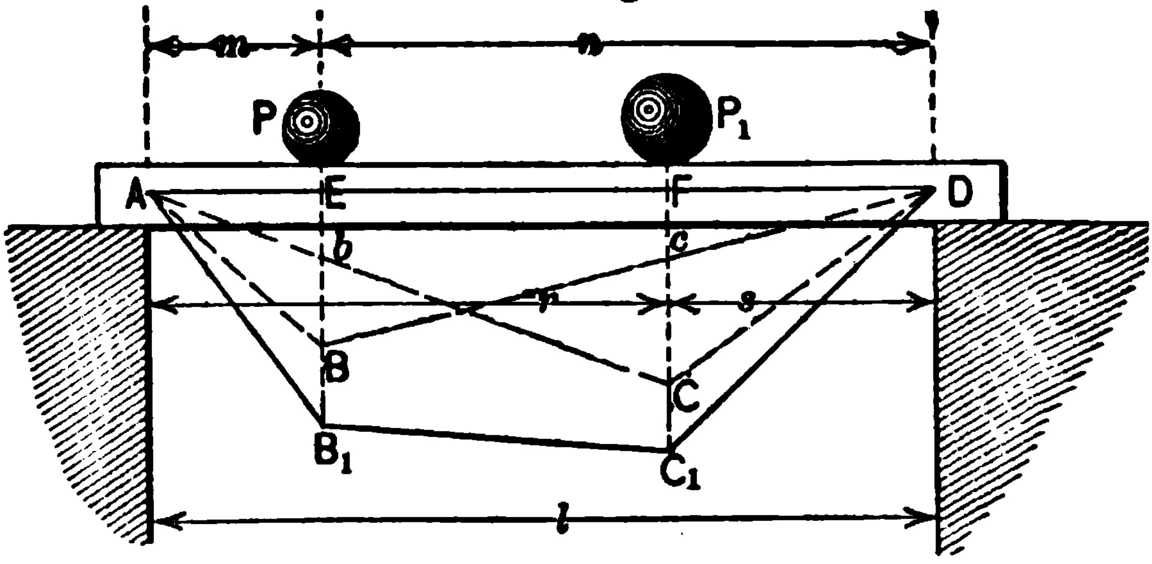

Fig. 14. Bending-moment Diagram. Two Concentrated Loads

Beam with Two Concentrated Loads (Fig. 14).

To draw the bending-moment diagram for a beam with two conce loads, draw the dotted lines ABD and ACD, giving the BENDING-MOME

...ms for each load separately. EB is laid out to scale, equal to Pmn/l and ...C equal to P_1rs/l.

The bending moment at the point E is equal to EB (from the load P) + Eb ...m the load P_1), or $M = EB + Eb = EB_1$; and at F the bending moment is ...ual to $FC + Fc = FC_1$. The BENDING-MOMENT DIAGRAM for both loads is ...CD and the MAXIMUM BENDING MOMENT is, in this particular case, the line ...C measured to scale.

...am with Any Number of Concentrated Loads (Fig. 15.)

Proceed as in the last case, and draw the BENDING-MOMENT DIAGRAM for each ...d separately. Make $AD = A1 + A2 + A3$, $BE = B1 + B2 + B3$ and

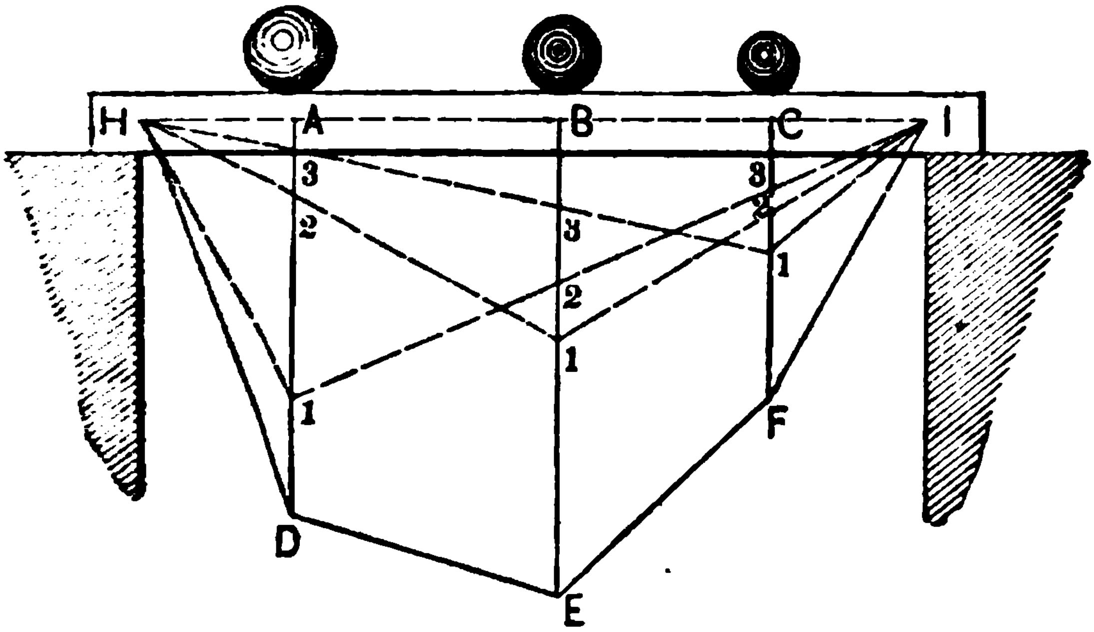

Fig. 15. Bending-moment Diagram. Three Concentrated Loads

...$= C1 + C2 + C3$. The figure $HDEFIH$ will then be the BENDING-MO...NT DIAGRAM corresponding to all the loads. The BENDING-MOMENT DIAGRAM ...a beam with any number of concentrated loads may be drawn in the same ...y.

...am with a Uniformly Distributed Load (Fig. 16).

...aw the beam with the given span, accurately to a scale as before, and at ...middle of the beam draw the vertical line AB, to a scale of a certain number ...FOOT-POUNDS to ... LINEAR INCH, ...al to $Wl/8$, from ...e V, W representing the whole distrib...d load. Connect ...points C, B, D by ...PARABOLA to obtain ...BENDING-MOMENT ...GRAM. To find ...bending moment ...ny point a, draw ...vertical line ab, ...sure it to the ...e scale to which AB is drawn, and it will be the bending moment desired. ...hods for drawing the PARABOLA will be found in Part I, page 79.

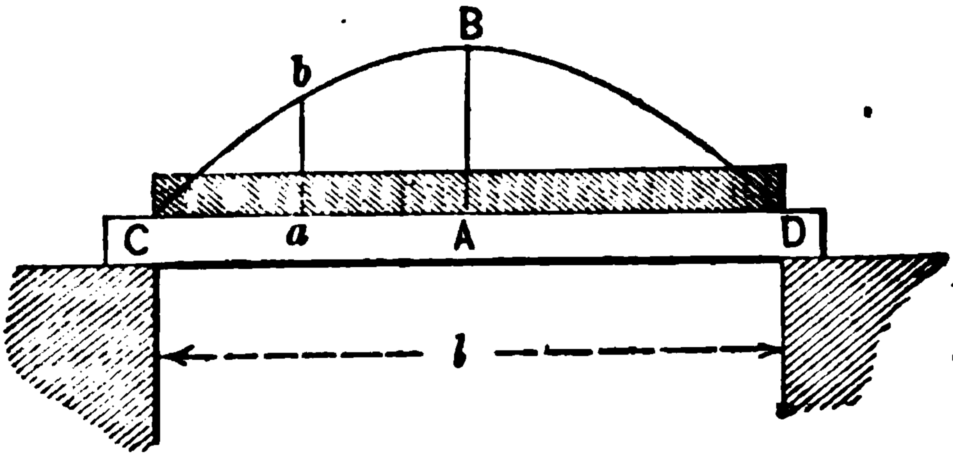

Fig. 16. Bending-moment Diagram. Distributed Load over Whole Beam

...am Loaded with Both Distributed and Concentrated Loads (Fig. 17).

...determine the bending moments in this case, combine the BENDING-MOMENT ...RAMS for the concentrated loads and for the distributed load, as shown in

Fig. 17. The bending moment at any section of the beam will then be lin by the line ABC on top and by the line $CDEFA$ on the bottom; and the 1 IMUM BENDING MOMENT will be the longest vertical line that can be dı

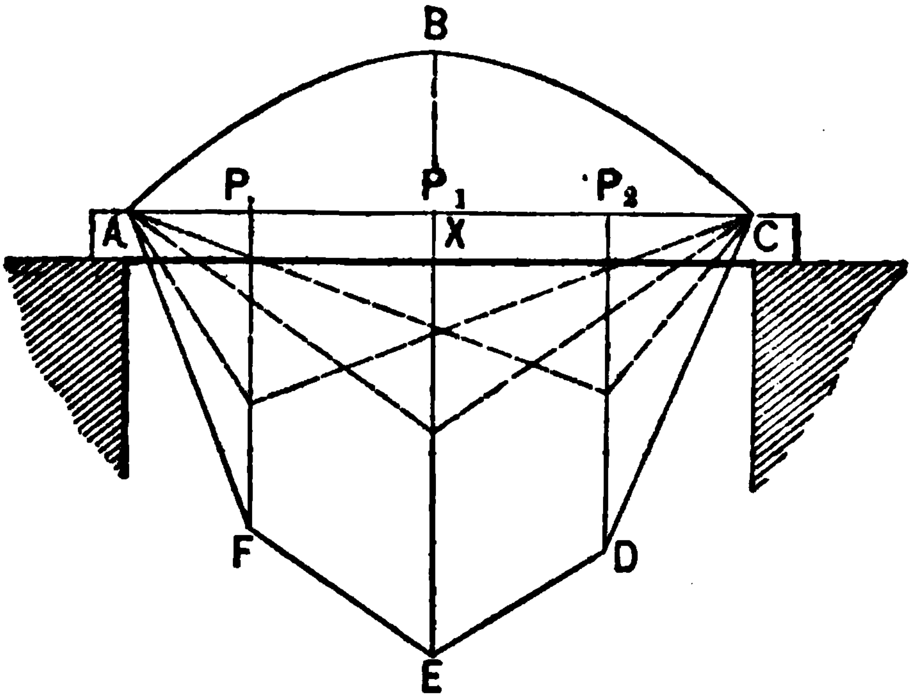

Fig. 17. Bending-moment Diagram. Distributed and Concentrated Loads

between these bounding lines.

For example, bending moment : is BE. The poi MAXIMUM BENDING MENT depends upo position of the co trated loads and relative magnitu the distributed it may or may occur at the mid the beam or unde of the concent loads.

Example 6. ' is the greatest be moment in a bea 20 ft span (Fig. 18), loaded with a distributed load of 800 lb, a concentrated l 500 lb 6 ft from one end, and a concentrated load of 600 lb 7 ft from the other

Solution. (1) The maximum bending moment due to the distributed from Case V, is $Wl/8$, or $800 \times 20/8 = 2\,000$ ft-lb. Lay off vertically ove middle of the beam, and at any convenient scale, say 4 000 ft-lb to the inch, $B1 = 2\,000$ ft-lb, and draw a parabola through the points A, B and C. (See page 79.)

(2) The maximum bending moment for the concentrated load of 500 lb, from Case VI, is $500 \times 6 \times 14/20$, or 2 100 ft-lb. Draw $E2 = 2\,100$ ft-lb to the same scale as $B1$, and then draw the lines AE and CE.

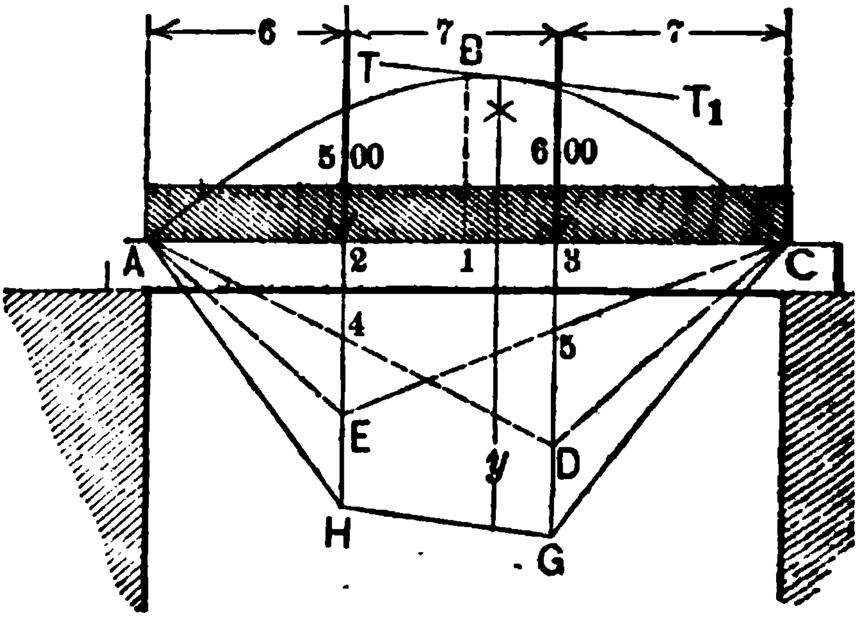

Fig. 18. Bending-moment Diagram. Distributed Concentrated Loads

(3) The maximum bending moment for the concentrated load of 600 like manner, is $600 \times 7 \times 13/20$, or 2 730 ft-lb. Draw $D3 = 2\,730$ ft-ll connect D with A and C.

(4) Make EH equal to the distance from 2 to 4, and DG to the distance 3 to 5, and draw $AHGC$.

The MAXIMUM BENDING MOMENT will be represented by the longest v line which can be drawn between the parabola ABC and the broken line A In this example the longest vertical line which can be drawn is Xy, should scale 5 645 ft-lb.

he position of the line Xy is determined by drawing the line TT_1 parallel C and tangent to ABC. Draw Xy vertically through point of tangency.

actions and Bending Moments for Beams with Triangular Loading and for Beams Fixed at Both Ends.*

ams with Triangular Loading have reactions and bending moments as ws:

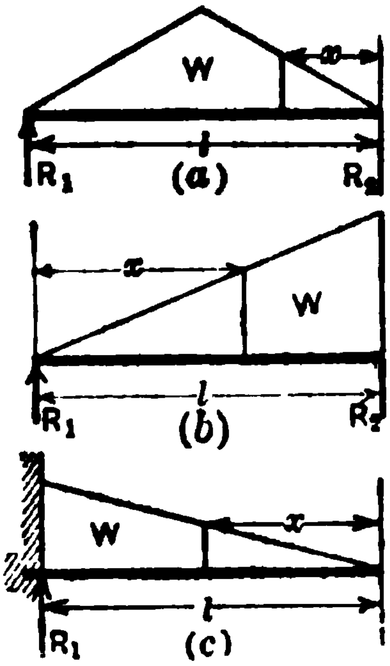

Fig. 19. Triangular Loading on Beams

Beam Supported at Both Ends, Fig. 19 (a)

eactions: $R_1 = R_2 = \frac{1}{2} W$
ing moment at any point $= Wx(\frac{1}{2} - 2x^2/3l^2)$
imum bending moment, at center $= Wl/6$

Beam Supported at Both Ends, Fig. 19 (b)

eactions: $R_1 = \frac{1}{3} W,\ R_2 = \frac{2}{3} W$
ing moment at any point $= (Wx/3)(1 - x^2/l^2)$
imum bending moment (at $x = 0.58\,l$) $= .128\,Wl$

Cantilever Beam, Fig. 19 (c)

tion: $R_1 = W$
ing moment at any point $= Wx^3/3\,l^2$
imum bending moment (at R_1) $= Wl/3$

ams of Cases IV, V, and VI, with Fixed Ends, e reactions and bending moments as follows:

IV A. Beam Fixed at Both Ends, with a Concentrated Load P at the Middle (Fig. 8)

End-reactions: $R_1 = R_2 = \frac{1}{2} P$
Maximum positive bending moment, under the load $= Pl/8$
Maximum negative bending moment, at ends $= Pl/8$

V A. Beam Fixed at Both Ends, with a Uniformly Distributed Load W (Fig. 9)

End-reactions: $R_1 = R_2 = \frac{1}{2} W$
Maximum negative bending moment, at ends $= Wl/12$
Maximum positive bending moment, at center $= Wl/24$

VI A. Beam Fixed at Both Ends, with a Concentrated Load P at Distance m from Left End and Distance n from Right End (Fig. 10)

End-reactions: $R_1 = Pn^2(3m + n)/l^3$; $R_2 = Pm^2(3n + m)/l^3$
Maximum bending moment, negative, at left end, $M_1 = Pmn^2/l^2$
at right end, $M_2 = Pm^2n/l^2$
Bending moment under load, positive $= R_1m - M_1$

* From notes by Robins Fleming.

CHAPTER X

PROPERTIES OF CROSS-SECTIONS OF STRUCTUR SHAPES. MOMENT OF INERTIA, MOMENT OF RESISTANCE, SECTION-MODULUS, AND RADIUS OF GYRATION

By

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1. The Properties of Cross-Sections

The Moment of Inertia. The strength of a cross-section to resist s in either a beam or a column, depends not only upon the area but als the form of the cross-section. The parts of the cross-section farthest fr neutral axis, which always passes through the center of gravity of the section, are much more efficient in resisting bending stresses than thos adjacent to the axis; so that some mathematical expression must be ol that will represent the efficiency of the entire cross-section to resist b stresses when compared with that of any other cross-section. This expre called the MOMENT OF INERTIA and is usually designated by the letter I.

The Moment of Inertia of any cross-section may be defined as the the products obtained by multiplying each of the elementary areas of wh section is composed by the square of its normal distance from the neut of the section.

By an ELEMENTARY AREA is meant an area smaller than any dealt simple mathematics, and it is, therefore, impossible to find an exact exp for the moment of inertia of a cross-section by such methods. By means calculus, however, exact formulas have been deduced from which the m of inertia of simple geometrical forms, such as rectangles, triangles, circl may be found, with respect to different axes.

The NEUTRAL AXIS of the cross-section of a beam, girder, column, etc., is in a state of flexure, is the line on which there is neither tension nor co sion in the fibers, and when the unit stresses do not exceed the ELASTIC L the material, it can be shown that this neutral axis passes through the CEN GRAVITY of the cross-section. The normal distance of the extreme fibers fr neutral axis is usually designated by the letter c or the letter y. The fo used in the notation of this book.

Since for all sections except squares and circles, there are, in general, tw tral axes corresponding to the more common positions of the sections, it that there are also two moments of inertia commonly used; for a rectan example, a GREATEST MOMENT OF INERTIA about an axis perpendicular long side and a LEAST MOMENT OF INERTIA about an axis perpendicular short side. The moments of inertia of the cross-sections of all rolled have been calculated and are tabulated in the manufacturers' handbooks. for example, the moments of inertia of the cross-section of a 12-in, 31.5-lb I with respect to axes perpendicular to the web and parallel to the web, ar Table IV, equal to 215.8 and 9.5 biquadratic inches respectively. Formu calculating the moments of inertia of other simple sections are given following pages.

he Moment of Resistance. In Chapter IX, under the chapter-subdivi-
treating of the BENDING MOMENTS in beams, page 325, it was stated that
rder to calculate the FLEXURAL STRENGTH of a beam it is necessary to
tain the nature and extent, first, of the external forces tending to break
beam by flexure, and, secondly, of the internal forces or stresses tending
sist rupture. The external forces cause the BENDING MOMENTS,* and the
nal stresses the MOMENTS OF RESISTANCE, at the various cross-sections of
beam.

he MOMENT OF RESISTANCE or the RESISTING MOMENT at any cross-section
beam is the algebraic sum of all the moments of the internal horizontal
es in that section with reference to a point in that section. It is usually
sented by the expression SI/c, in which S is the horizontal unit stress,
e or compressive, as the case may be, upon the fiber most remote from the
ral axis of the section, and called the FIBER-STRESS; I is the MOMENT OF
RTIA of the area of the section with reference to the NEUTRAL AXIS; and c
he shortest distance from the most remote fiber to that axis. Since, for
ibrium of forces and stresses at any cross-section of a beam, the bending
ent equals the resisting moment for that section, if M represents the bend-
moment we have the equation

$$M = SI/c \tag{1}$$

his is known as the FLEXURE FORMULA and is universally used for investi-
ing the flexural strength of beams.

he Section-Modulus or Section-Factor. That expression I/c in the above
ula is generally known as the SECTION-MODULUS or SECTION-FACTOR.
quantity for the principal rolled sections is given in Tables IV, V, VI, VII,
I, XI, XII, XIII and XIV. Corresponding to the two moments of inertia
ally used for all sections (except for squares and circles) there are two
n-moduli also, one for each axis. Thus, the section-modulus of the 12-in
lb I beam, with respect to a neutral axis perpendicular to the web, is
$= 215.8/6 = 36$; and for the axis parallel to the web, it is $I/c = 9.5/2.5 = 3.8$.
other shapes the section-modulus may be found by dividing the moment of
tia by the normal distance of the extreme fiber from the neutral axis.

he Radius of Gyration. The effect of the form of the cross-section of a
mn on its strength is determined by a quantity called the RADIUS OF GYRA-
I, which is as necessary in the determination of the strength of a column as
moment of inertia is in the determination of the strength of a beam. It is
ted by the letter r. The value of the radius of gyration for any section
termined by the formula

$$r = \sqrt{I/A} \tag{2}$$

hich I is the MOMENT OF INERTIA of the section and A the SECTION-AREA.
RADIUS OF GYRATION is the normal distance from the NEUTRAL AXIS to the
ER OF GYRATION, and the center of gyration of a section is the point where
ntire area might be concentrated and have the same moment of inertia
e actual distributed area. The radius of gyration of a section is a DISTANCE
h is always less than the distance, c, from the neutral axis to the remotest
. For the two moments of inertia above referred to, and commonly used,
are two corresponding radii of gyration. The least of these is the one to be
in the investigation of the strength of a column as it is referred to the axis
t which the column is most likely to fail. The radii of gyration of the rolled

* See Chapter IX, page 325, for definition of "bending moment."

shapes are given in the tables of the properties of sections, mentioned a For the 12-in 31.5-lb I beam, $r = 4.83$ in and $r' = 1.01$ in. The radius of gyı of any other section may be found by Formula (2).

Formulas for the moments of inertia, radii of gyration and section-m of the principal elementary sections are given on the following pages.] case of a hollow section or a section with a reentering hollow part, the mc of inertia of the hollow part is to be subtracted from that of the enclosing Moments of inertia when referred to the same axis can be added or subtı like any other quantities which are of the same kind.

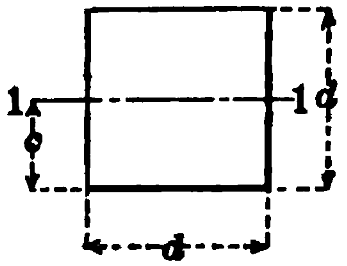
2. Areas, Moments of Inertia, Section-Moduli and Radii of Gyration of Elementary Sections

I = the moment of inertia
I/c = the section-modulus
r = the radius of gyration
A = the area of the section
c = the normal distance of most remote fiber from neutral axi

The position of axis referred to in each case is represented by the broke 1 — 1.

SQUARE

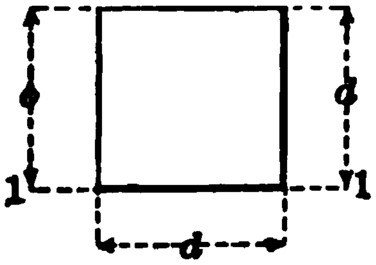
Axis of moments through center



$$A = d^2$$
$$c = \frac{d}{2}$$
$$I = \frac{d^4}{12}$$
$$\frac{I}{c} = \frac{d^3}{6}$$
$$r = \frac{d}{\sqrt{12}} = 0.288675\,d$$

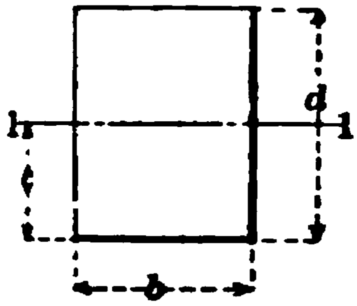
SQUARE

Axis of moments on base



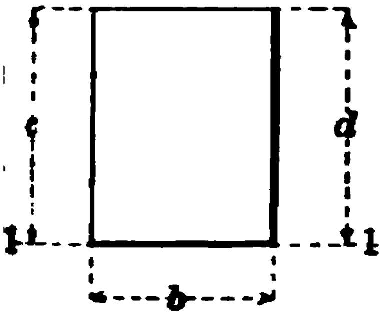
$$A = d^2$$
$$c = d$$
$$I = \frac{d^4}{3}$$
$$\frac{I}{c} = \frac{d^3}{3}$$
$$r = \frac{d}{\sqrt{3}} = 0.577350\,d$$

RECTANGLE
his of moments through center



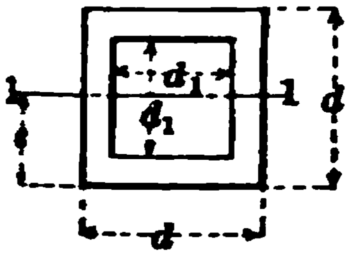
$$A = bd$$
$$c = \frac{d}{2}$$
$$I = \frac{bd^3}{12}$$
$$\frac{I}{c} = \frac{bd^2}{6}$$
$$r = \frac{d}{\sqrt{12}} = 0.288675\,d$$

RECTANGLE
his of moments on base



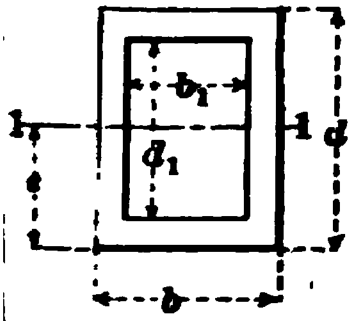
$$A = bd$$
$$c = d$$
$$I = \frac{bd^3}{3}$$
$$\frac{I}{c} = \frac{bd^2}{3}$$
$$r = \frac{d}{\sqrt{3}} = 0.577350\,d$$

HOLLOW SQUARE
his of moments through center



$$A = d^2 - d_1^2$$
$$c = \frac{d}{2}$$
$$I = \frac{d^4 - d_1^4}{12}$$
$$\frac{I}{c} = \frac{d^4 - d_1^4}{6\,d}$$
$$r = \sqrt{\frac{d^2 + d_1^2}{12}}$$

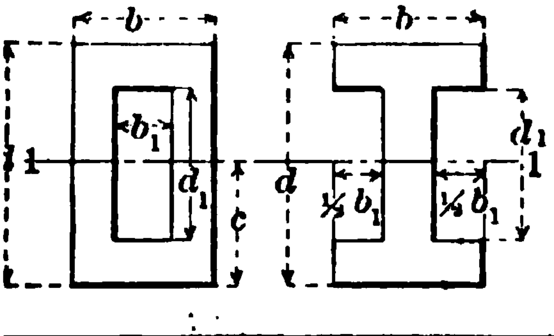
LLOW RECTANGLE
s of moments through center



$$A = bd - b_1d_1$$
$$c = \frac{d}{2}$$
$$I = \frac{bd^3 - b_1d_1^3}{12}$$
$$\frac{I}{c} = \frac{bd^3 - b_1d_1^3}{6\,d}$$
$$r = \sqrt{\frac{bd^3 - b_1d_1^3}{12\,(bd - b_1d_1)}}$$

HOLLOW RECTANGLE AND I BEAM

Axis of moments through center



$$A = bd - b_1d_1$$

$$c = \frac{d}{2}$$

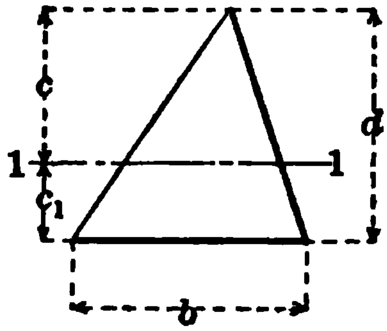
$$I = \frac{bd^3 - b_1d_1^3}{12}$$

$$\frac{I}{c} = \frac{bd^3 - b_1d_1^3}{6\,d}$$

$$r = \sqrt{\frac{bd^3 - b_1d_1^3}{12\,(bd - b_1d_1)}}$$

TRIANGLE

Axis of moments through center of gravity



$$A = \frac{bd}{2}$$

$$c = \frac{2\,d}{3} \qquad c_1 = \frac{d}{3}$$

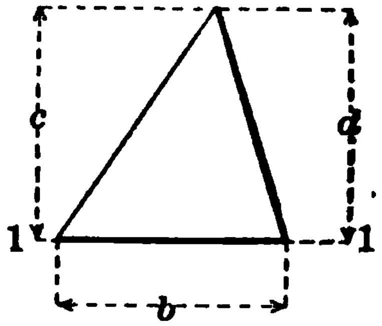
$$I = \frac{bd^3}{36}$$

$$\frac{I}{c} = \frac{bd^2}{24}$$

$$r = \frac{d}{\sqrt{18}} = 0.235702\,d$$

TRIANGLE

Axis of moments on base



$$A = \frac{bd}{2}$$

$$c = d$$

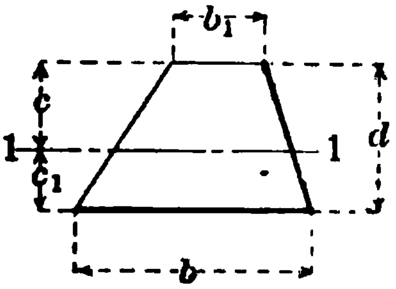
$$I = \frac{bd^3}{12}$$

$$\frac{I}{c} = \frac{bd^2}{12}$$

$$r = \frac{d}{\sqrt{6}} = 0.408248\,d$$

TRAPEZOID

Axis of moments through center of gravity



$$A = \frac{d\,(b + b_1)}{2}$$

$$c^* = \frac{d\,(b_1 + 2\,b)}{3\,(b + b_1)} \qquad {}^*c_1 = \frac{d\,(b +}{3\,(b -}$$

$$I = \frac{d^3\,(b^2 + 4\,bb_1 + b_1^2)}{36\,(b + b_1)}$$

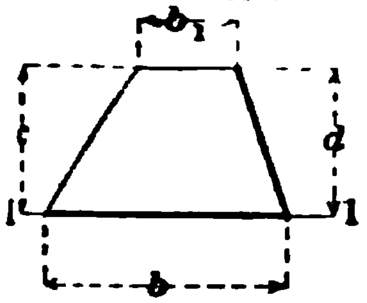
$$\frac{I}{c} = \frac{d^2\,(b^2 + 4\,bb_1 + b_1^2)}{12\,(b_1 + 2b)}$$

$$r = \frac{d}{6\,(b + b_1)}\sqrt{2\,(b^2 + 4bb_1}$$

* To find c and c_1, see Chapter VI, page 295.

TRAPEZOID

Axis of moments on base



$$A = \frac{d(b + b_1)}{2}$$

$$c = d$$

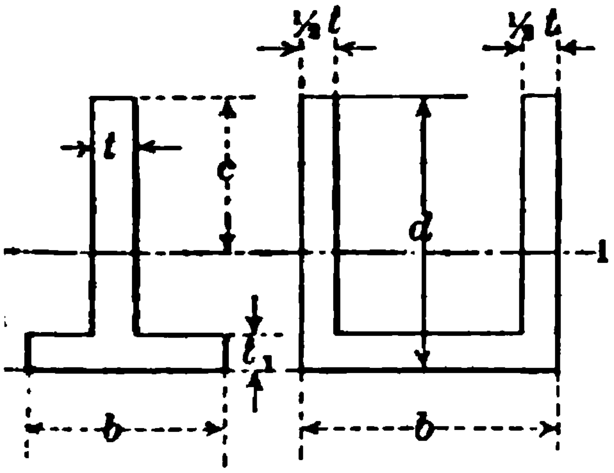
$$I = \frac{d^3(b + 3b_1)}{12}$$

$$\frac{I}{c} = \frac{d^2(b + 3b_1)}{12}$$

$$r = \frac{d}{\sqrt{6}}\sqrt{\frac{b + 3b_1}{b + b_1}}$$

T SECTION AND CHANNEL

Axis of moments through center of gravity



$$A = td + t_1(b - t)$$

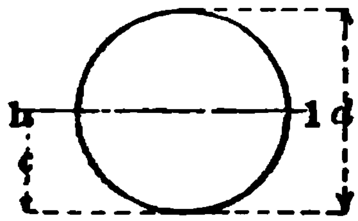
$$c^* = \frac{td \times \frac{1}{2}d + t_1(b - t)(d - \frac{1}{2}t_1)}{A}$$

$$I = \frac{tc^3 + bc_1^3 - (b - t)(c_1 - t_1)^3}{3}$$

$$r = \sqrt{\frac{I}{A}}$$

CIRCLE

Axis of moments through center



$$A = \frac{\pi d^2}{4} = 0.785398\,d^2$$

$$c = \frac{d}{2}$$

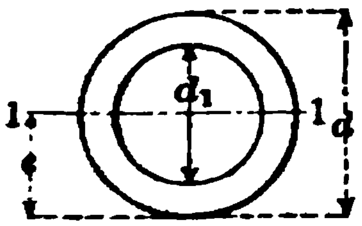
$$I = \frac{\pi d^4}{64} = 0.049087\,d^4$$

$$\frac{I}{c} = \frac{\pi d^3}{32} = 0.098175\,d^3$$

$$r = \frac{d}{4}$$

HOLLOW CIRCLE

Axis of moments through center



$$A = \frac{\pi(d^2 - d_1^2)}{4} = 0.785398\,(d^2 - d_1^2)$$

$$c = \frac{d}{2}$$

$$I = \frac{\pi(d^4 - d_1^4)}{64} = 0.049087\,(d^4 - d_1^4)$$

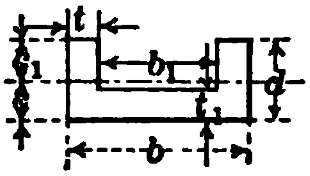
$$\frac{I}{c} = \frac{\pi(d^4 - d_1^4)}{32\,d} = 0.098175\,\frac{(d^4 - d_1^4)}{d}$$

$$r = \frac{\sqrt{d^2 + d_1^2}}{4}$$

* To find the values of c and c_1, see Chapter VI, page 295.

CHANNEL

Axis of moments through center of gravity



$$A = t_1b + 2t(d - t_1)$$

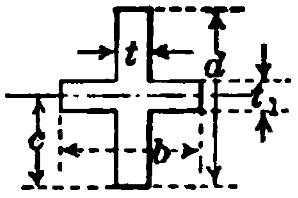
$$c^* = \frac{2d^2t + b_1t_1^2}{2A}$$

$$I = \frac{2td^3 + b_1t_1^3}{3} - Ac^2$$

$$r = \sqrt{\frac{I}{A}}$$

CROSS-SECTION

Axis of moments through center of gravity



$$A = td + t_1(b - t)$$

$$c = \frac{d}{2}$$

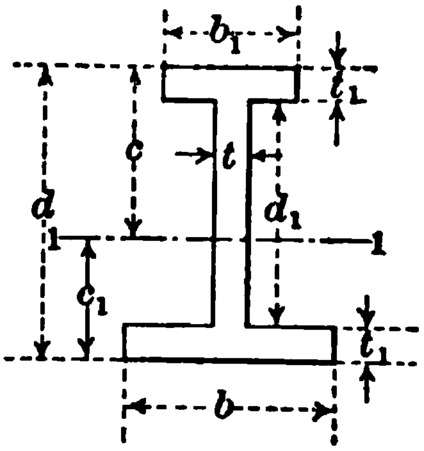
$$I = \frac{td^3 + t_1^3(b - t)}{12}$$

$$\frac{I}{c} = \frac{td^3 + t_1^3(b - t)}{6d}$$

$$r = \sqrt{\frac{td^3 - t_1^3(b - t)}{12(td + t_1(b - t))}}$$

IRREGULAR I SHAPE

Axis of moments through center of gravity



$$A = bt_1 + d_1t + b_1t_1$$

$$c^* = \frac{td^2 + t_1^2(b_1 - t) + t_1(b - t)(2d -}{2A}$$

$$c_1 = \frac{td^2 + t_1^2(b - t) + t_1(b_1 - t)(2d -}{2A}$$

$$I = \frac{b_1c^3 - (b_1 - t) \times (c - t_1)^3}{3} + \frac{bc_1^3 - (b - t) \times (c_1 - t_1)^3}{3}$$

$$r = \sqrt{\frac{I}{A}}$$

* To find c and c_1, see Chapter VI, page 295.

3. Transferring Moments of Inertia to Other Parallel Axes

Explanation of Formula. It is often necessary to determine the m of inertia with respect to some other axis than the one passing throu center of gravity of the section, such, for example, as one passing throu base and parallel to the other. Suppose it is desired to find the moment ertia of a rectangle about an axis passing through the lower base, as second figure on page 335. It may be demonstrated by the princi mechanics that the moment of inertia of any section with respect to an is equal to the moment of inertia of the section with respect to a parall through the center of gravity, plus the product of the area of the section plied by the square of the normal distance between the axes. This rule r expressed by the formula

$$I_1 = I + Ah^2$$

wich I_1 is the required moment of inertia, I the moment of inertia of the tion with respect to the axis through its center of gravity and parallel to the n axis, A the area of the section and h the normal distance between the axes. m this it is seen that the moment of inertia of any section-area is less for axis through its center of gravity than for any other parallel axis.

For example, consider the rectangle shown on page 335, of breadth b and th d, the I of which is known to be $bd^3/12$ for an axis passing through the ter of gravity and parallel to the base. Then, for allel axis through the base, the above formula es

$$I_1 = \frac{bd^3}{12} + bd \times \left(\frac{d}{2}\right)^2 = \frac{bd^3}{12} + \frac{bd^3}{4} = \frac{bd^3}{3}$$

hus the moment of inertia of the cross-section he steel angle shown in Fig. 1, about the axis N, is equal to the moment of inertia about the s XX plus the product of its area multiplied h^2. The moments of inertia for the sections of standard rolled shapes of structural steel may found from the tables given in this chapter. The ance c_1, also, may be found from the same tables; this distance subtracted from d will give the ance h of Formula (3).

Suppose, for example, that it is desired to find the ment of inertia of the cross-section of a 4 by 3 in angle, placed, with the long leg horizontal, t an axis MN, 12 in from the back (Fig. 1). Turning to Table XI, the of the angle-section = 3.25 sq in. I, the moment of inertia of the angle-ion about an axis 2–2, or XX of Fig. 1, parallel to the long leg = 2.4, c_1, the ance of this axis from the back of the long leg = 0.83 in and h, the distance een the axes = $(d - c_1)$ = 12 − 0.83 in = 11.17 in. Substituting these values ormula (3)

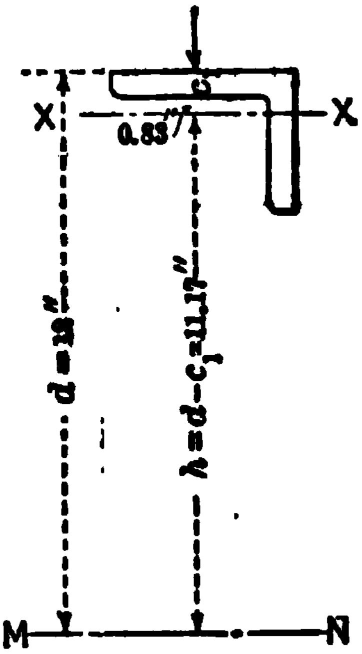


Fig. 1. Moment of Inertia of Cross-section of Steel Angle.

$$I_1 = 2.4 + 3.25 \times 11.17^2 = 2.4 + 405.50 = 407.9$$

4. Moments of Inertia of Compound Sections

he Moment of Inertia of a Compound Section made up of a number of ler sections may be found by the same formula, $I_1 = I + Ah^2$. Denote the OF THE MOMENTS OF INERTIA of the separate sections making up the com-d section, with respect to an axis through the center of gravity of that m, by ΣI_1. Formula (3) then becomes

$$\Sigma I_1 = \Sigma\,(I + Ah^2) \qquad (4)$$

at is, to find the moment of inertia of any compound section made up of mber of smaller sections:

Find the moment of inertia of each of the smaller sections about an axis g through its own center of gravity and parallel to the neutral axis of the ound section;

Multiply the area of each of the smaller sections by the square of the dis-between its center of gravity and the center of gravity of the whole figure;

Add the results found by (1) and (2) for the moment of inertia of the e figure.

r example, consider the cast-iron beam or lintel shown in section in Fig. 2:

(1) I of upper flange-section $= 4 \times 1^3/12 = 4/12$
I of web-section $= 1 \times 18^3/12 = 5\,832/12$
I of lower flange-section $= 16 \times 1^3/12 = 16/12$
Total $= 5\,852/12 = 487.6$

(2) Ah^2 for the upper flange $= 4 \times (12.5)^2 = 625$
Ah^2 for the web $= 18 \times 3^2 = 162$
Ah^2 for the lower flange $= 16 \times (6.5)^2 = 676$
Total $= 1\,463$

(3) Total of (1) and (2) $= 487.6 + 1\,463 = I_1$ of compound section $= 1\,9$

The moment of inertia of the cross-section of any compound beam, then can generally be readily found by using the tables of properties of sections

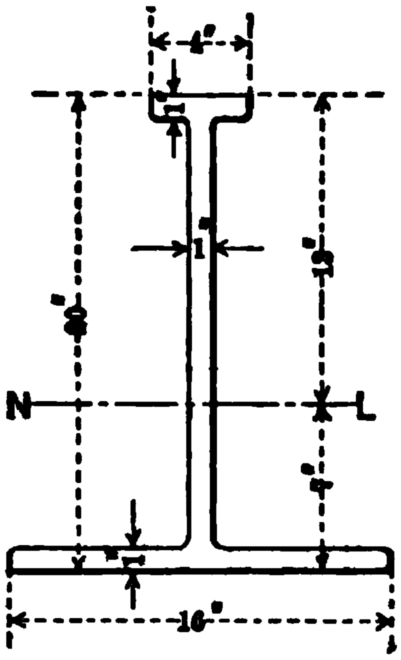


Fig. 2. Moment of Inertia of Cross-section of Cast-iron Lintel

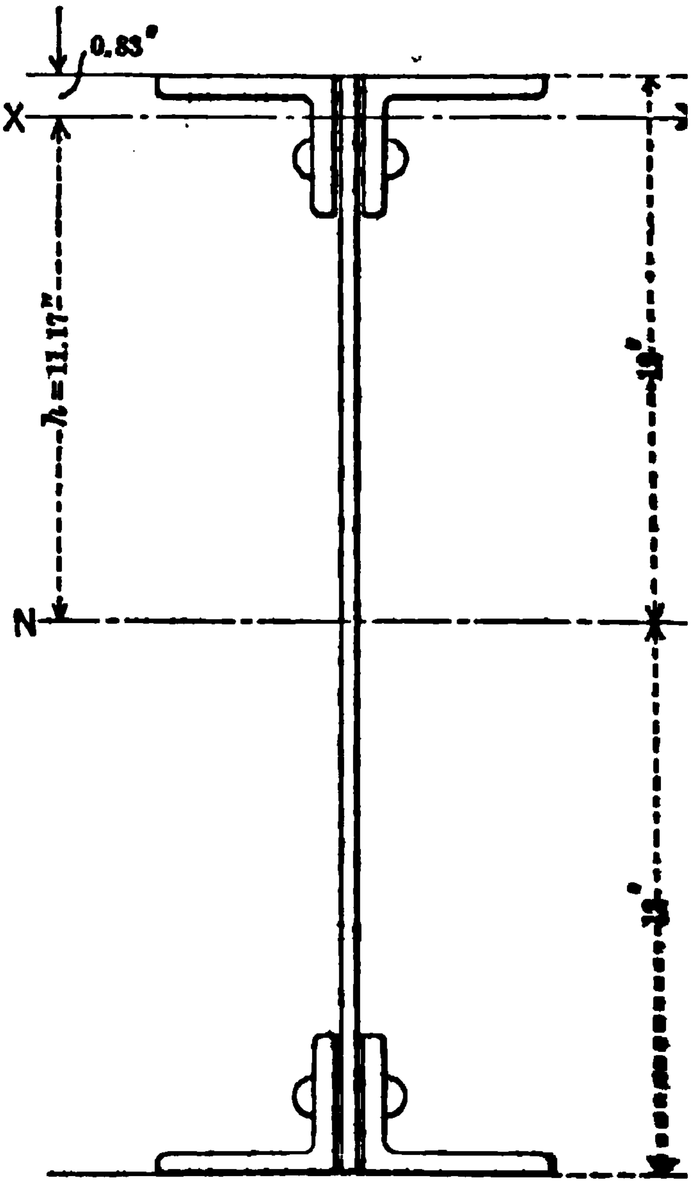


Fig. 3. Moment of Inertia of Cross-sect of Plate Girder. No Flange-plates

give the numerical values of I for the various rolled shapes of which the beam is composed, with respect to the axis through the center of gravity.

The Moment of Inertia of a Single-Web Girder-Section. Consider, for example, the single-web girder shown in section in Fig. 3, and made up of one ½ by 24-in web and four 4 by 3 by ½-in flange-angles with the long legs placed horizontally. ing to Table XI, the moment of inertia of the cross-section of o these angles about an axis XX (2–2 in the table) parallel to the leg = 2.4, and the distance of this axis from the back of the long leg (y table) = 0.83 in; hence h, the distance between the axis of the angle-s and the axis of the girder-section = 12 − 0.83 = 11.17 in. A, from the = 3.25 sq in. The moment of inertia of the cross-section of each angle the axis of the girder, therefore, from Formula (3), is $I_1 = 2.4 + 3.25 \times (11$ 407.9, and for the four angles = 1631.6. Since the axis of the cross-sect

web-plate is coincident with the axis of the section of the girder, its moment of inertia = $bd^3/12$ = $\frac{1}{2} \times (24)^3/12 = 576$. This may be found directly from Table I, page 346, Moments of Inertia of Rectangles. The moment of inertia, therefore, of the section of the compound girder = 1631.6 + 576 = 2207.6.

The Moment of Inertia of a Section of a Compound Girder with Flange-Plates is found in the same way, except that the moments of inertia of the sections of the flange-plates with respect to the axis of the girder-section must be added to the moments of inertia of the cross-sections of the other members. The girder in Fig. 4 is composed of one 30 by 3/8-in web-plate, four 6 by 4 by 9/16-in angles, with the longer legs horizontal, and two 12 by ½-in flange-plates.

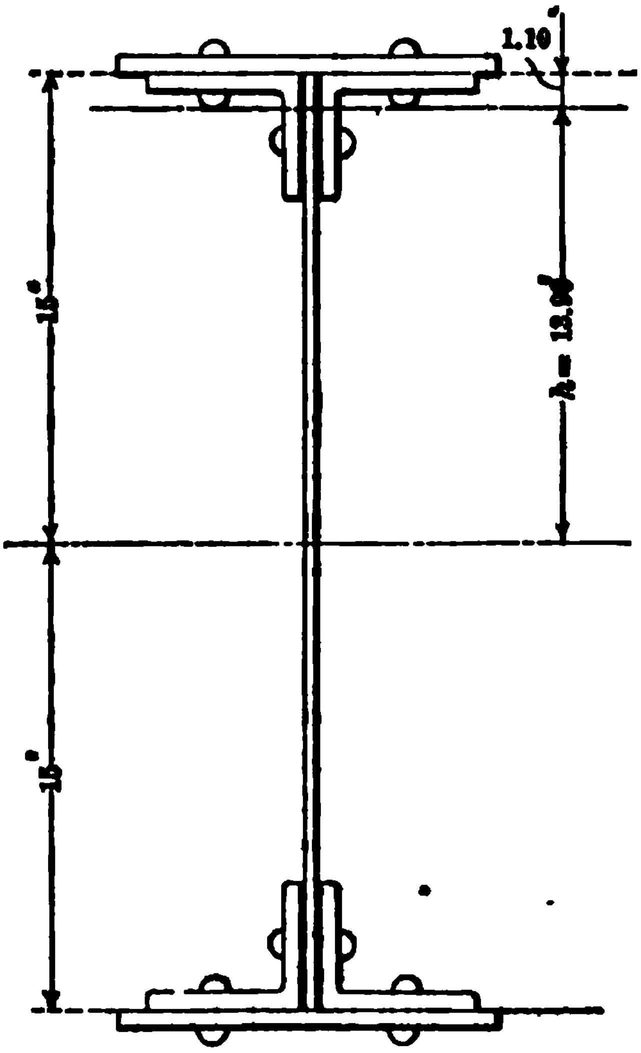


Fig. 4. Moment of Inertia of Cross-section of Plate Girder with Flange-plates

I (= I_1) for cross-section of web (from Table I, page 346) = 843.75

I_1 for each angle-section = $I + Ah^2$ (Formula 3)

From Table XI, for each flange-angle, I = 6.6, A = 4.75 and the perpendicular distance from center of gravity to back of long leg = 1.10 in. Hence $h = 15 - 1.10 = 13.90$ in. $I_1 = 6.6 + 4.75 \times (13.90)^2 = 924.35$; and for four angles 3697.4. I for the cross-section of each flange-plate = $12 \times (\frac{1}{2})^3/12 = 0.125$, $A = \frac{1}{2} \times 12 = 6$ sq in and $h = 15 + \frac{1}{4} = 15.25$ in. For each flange-plate, then, $I_1 = 0.125 + 6 \times (15.25)^2 = 1395.125$; and for the two plates, 2 790.25. The moment of inertia for the cross-section of the whole girder, therefore, with reference to the horizontal axis passing through the center of gravity of the section = 843.75 + 3 697.4 + 2 790.25 = 7 331.4.

It will be noticed that the moments of inertia of the cross-sections of the flange-plates and angles about their own neutral axes is so small, compared with their moments of inertia about the neutral axis of the girder-section, that they might be omitted without any appreciable error. Therefore, in calculating the moments of inertia for riveted girders, it is the custom of many engineers to let $I_1 = Ah^2$ for flange-plate and angle-sections. In that case, for the girder-section in Fig. 4,

I for web	= 843.75
I_1 for angles = Ah^2	= 3 671.00
I_1 for flange-plates = Ah^2	= 2 790.00
Moment of inertia of entire girder-section	= 7 304.75

The Moment of Inertia of a Section of a Box Girder. Let the box girder shown in Fig. 5 be composed of two 3/8 by 30-in webs, two 16 by ½-in plates and four 4 by 3 by ½-in angles with the long legs horizontal.

I for each flange-plate $= bd^3/12 = 16 \times (\frac{1}{2})^3/12 = 0.16$; $A = \frac{1}{2} \times 16$ in $= 8$ and $h = 15 + \frac{1}{4} = 15.25$ in. $I_1 = I + Ah^2 = 0.16 + 8 \times (15.25)^2 = 1\,860.64$; for the two flange-plates, 3 721.28. I for angle $= 2.4$, $A = 3.25$ and the distance fro back of the long leg to an axis throug center of gravity of the angle, parallel t long leg $= 0.83$ in; so that $h = 15 - 0.83 =$ in. I_1 for the four angles is $(4 \times 2.4) +$ $3.25) \times (14.17)^2 = 2\,619$. I for each web (I, page 346) $= 843.75$ and for the two $= 1\,687.5$. The moment of inertia, therefore, for the entire girder-section $=$ $3\,721.28 + 2\,619 +$ $1\,687.5 = 8\,027.78$.

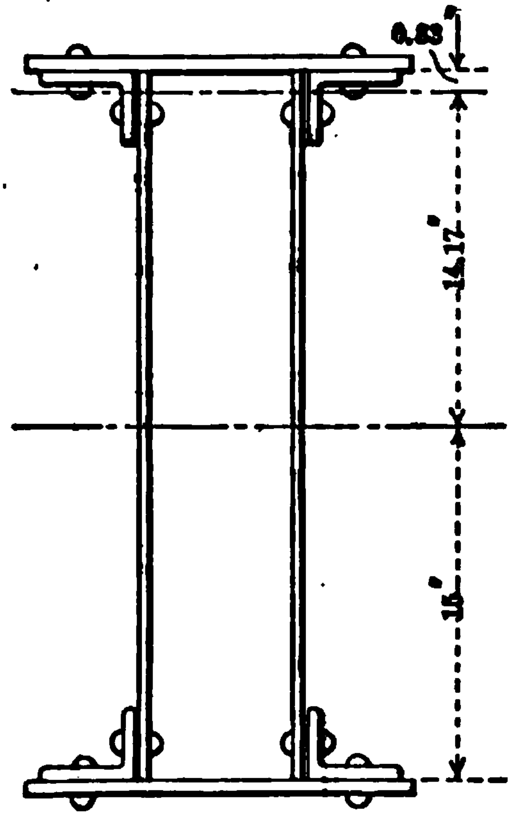


Fig. 5. Moment of Inertia of Cross-section of Plate-and-angle Box Girder

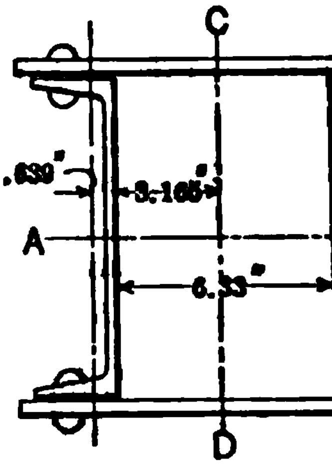


Fig. 6. Moment of of Cross-section o and-channel Box C

The Moment of Inertia of the Section of a Channel Box Column. Fig. 6 shows the cross-section of a column made up of two 10-in 15.3-lb channels, set 6.33 in apart, back to back, and two ½ by 12-in side plates. Let it be required to fin moment of inertia of the section about the two axes AB and CD.

(1) Find the moment of inertia about the axis AB. I, for one of t plates with respect to an axis through its own center of gravity and par $AB = 12 \times (\frac{1}{2})^3/12 = 0.125$, $A = \frac{1}{2} \times 12$-in $= 6$ sq in and the distance of its center of gravity from AB is 5.25 in. Therefore, with respect to AB, $I_1 = 0.125 + 6 \times (5.25)^2$ $= 165.5$. The moment of inertia of a 10-in 15.3-lb channel with respect to an axis through its center of gravity and perpendicular to the web (Table VIII, page 359) $= 66.9$. Hence the moment of inertia of the whole column-section with respect to the axis AB $= (2 \times 165.5) + (2 \times 66.9)$ $= 464.8$.

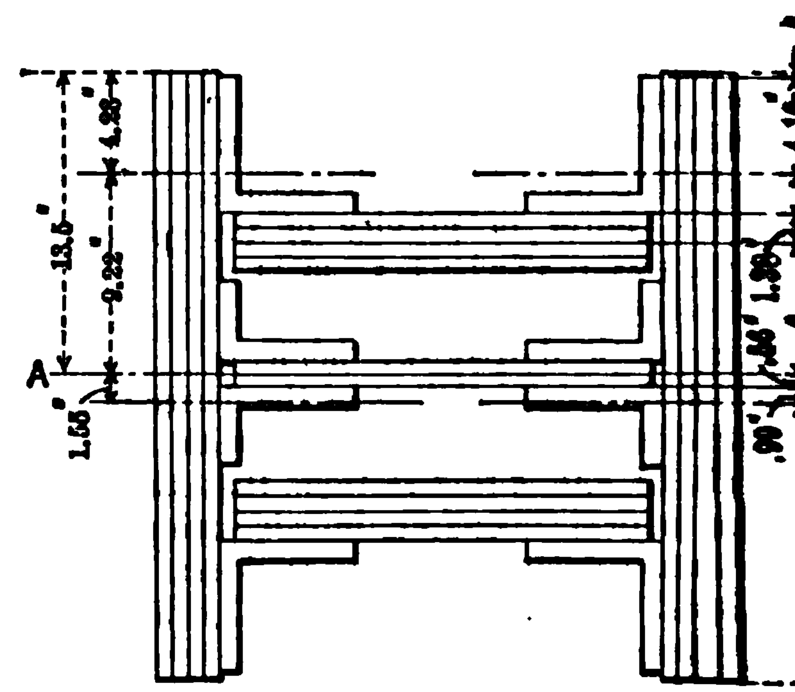


Fig. 7. Moment of Inertia of Cross-section of web Plate-and-angle Box Column

(2) Find the moment of inertia about the axis CD. I, for one of the side plates (Table 346) $= 72$. I, for one of the channels with respect to an axis parallel web $= 2.30$, $A = 4.47$ and the distance of the center of gravity from t of the web $= 0.64$ in, approximately. Hence $h = 3.165 + 0.64 =$ $I_1 = 2.30 + 4.47 \times (3.8)^2 = 66.8$ and the moment of inertia of the whole section with respect to the axis $CD = (2 \times 72) + (2 \times 66.8) = 277.6$.

he Moment of Inertia of the Section of a Heavy Plate-and-Angle mn. Fig. 7 shows the cross-section of one of the basement-columns in Bankers' Trust Company Building, New York City. It is made up of six e-plates, each 27 by ¾ in section; two flange-plates, each 27 by 1 1/16 in; r flange-angles, each 6 by 6 by 15/16 in; eight outer web-plates, each 18 by in; four web-angles, each 6 by 3½ by 15/16 in; and two middle web-plates h 18 by 9/16 in. What is its moment of inertia of the entire column-section h respect to the axis AB?

I for each 27 by ¾-in flange-plate (Table I) = 1 230.19	
I for six 27 by ¾-in flange-plates = 1 230.19 × 6 =	7 381.14
I for each 27 by 1 1/16-in flange-plate (Table I) = 1 127.67	
I for two 27 by 1 1/16-in flange-plates = 1 127.67 × 2 =	2 255.34
I for both flanges	9 636.48

or the flange-angles (Table XII, page 366) the area of a 6 by y 15/16-in angle = 10.37, its I with respect to an axis parallel to (Fig. 7) and passing through its center of gravity = 33.7 and distance of this axis from the back of the leg = 1.84 in. Its I_1 h respect to the axis AB is found by Formula (3), page 338, $I + Ah^2$. $h = 13.5 - (0.12 + 4.16)$ = the distance from the axis to the parallel axis through the center of gravity of the angle 22 in. Hence, substituting in Formula (3),

$$I_1 = 33.7 + 10.37 \times (9.22)^2 = 915.15$$

I_1 for the four flange-angles = 915.15 × 4 = 3 660.60

ach outer web is 4 × 11/16 in = 2¾ in thick. Hence the I for outer web about the horizontal axis through its center of ity = $18 \times (2.75)^3/12$ = 31.2. A = 18 by 2.75 in = 49.5 sq in. distance from its center of gravity to the axis AB is 13.5 − (1.38 + 4.16 + 0.12) = 6.01 or, say 6 in.

m Formula (3), therefore, $I_1 = 31.2 + 49.5 \times 6^2 = 1\,813.2$ and both outer webs $I_1 = 1\,813.2 \times 2$ = 3 626.4

r the four web-angles, from Table XI, page 363, the area of a 3½ by 15/16-in angle = 8.03, its I with respect to an axis through ter of gravity and parallel to the long leg = 6.9 and the dis- e of this axis from the back of the long leg = 0.99 in. h, the total distance between the two axes = 9/16 in, or 0.5625 in thickness of one of the middle web-plates) + 0.99 = 1.55 in, ximately. Therefore, for one web-angle, from Formula (3),

$$I_1 = 6.9 + 8.03 \times (1.55)^2 = 26.17$$

for the four angles, $I_1 = 26.17 \times 4$ = 104.68

e middle web-plates are together 9/16 in × 2 = 1⅛ in = 1.125 ck. The I (= I_1) for the two plates is $18 \times (1.125)^3/12$ = 2.14

e moment of inertia of the entire column-section for the axis , therefore, the sum of these moments of inertia for the differ- arts:

I_1 for the eight flange-plates	9 636.48
I_1 for the four flange-angles	3 660.60
I_1 for the eight outer web-plates	3 626.40
I_1 for the four web-plates	104.68
I_1 for the middle web-plates	2.14
The moment of inertia for the entire section	17 030.30

5. Radii of Gyration of Compound Sections

The Radius of Gyration of any Compound Section may be found f Formula (2), page 333, by dividing the moment of inertia of the section by total area of the section and taking the square root of the quotient. T the radii of gyration of the channel-column section shown in Fig. 6, about axes AB and CD, are found as follows: $A =$ (the sum of the areas of two 12-in plates, or 12 sq in) + (the sum of the areas of the two channels, or sq in) = 20.94 sq in. I about $AB = 464.8$ and about $CD = 277.6$.

Therefore, r, with respect to the axis $AB = \sqrt{\dfrac{464.8}{20.94}} = 4.71$

and r_1, with respect to the axis $CD = \sqrt{\dfrac{277.6}{20.94}} = 3.64$

Since r_1 is the smaller, it is the value to be used in the column-formula. to be noted that this value of r agrees with the r of the 10-in channel-colu Table XXV, on page 533. The value of r_1 does not, however, agree exactly the r_1 in the same table, the variation being caused by a difference in the sp of the channels, back to back.

The Least Radius of Gyration of a Section of a Plate-and-A Column. As another example, let it be required to find the least radi gyration of the cross-section of the plate angle column shown in Fig. 8, made up o 3/8 by 12-in web-plate, two 3/8 by 12-in side and four 4 by 4 by 1/2-in angles.

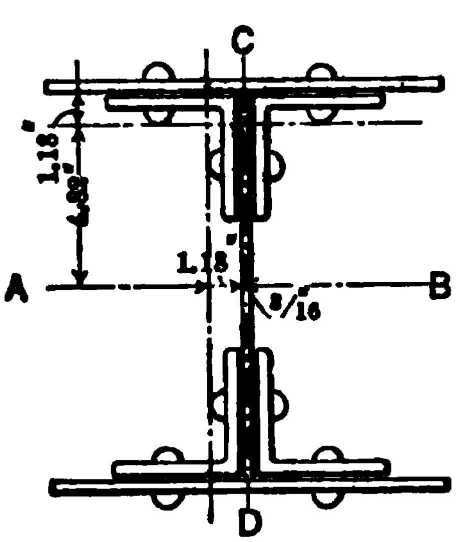


Fig. 8. Least Radius of Gyration of Cross-section of Plate-and-angle Column

(1) Find the moment of inertia about th AB. For the axis AB, I for each one of th plates with respect to an axis through its center of gravity and parallel to the axis $12 \times (3/8)^3/12 = 0.05$. $A = 3/8 \times 12 = 4.5$ sq $h = 6 3/16$ in. $I_1 = 0.05 + 4.5 \times (6 3/16)^2 =$ 1 I for each one of the angles with respect axis through its center of gravity and p with the flange-leg is 5.6, $A = 3.75$ and th tance of the center of gravity from the ba the flange of the angle = 1.18. Hence, h − 1.18 in = 4.82 in and h for each angle + $3.75 \times (4.82)^2 = 92.71$. I for the web-p 54. The moment of inertia of the whole column-section, therefore, abo axis $AB = (2 \times 172.33) + (4 \times 92.71) + 54 = 769.50$.

(2) Find the moment of inertia about the axis CD. I for each side = 54. I for each angle = 5.6 and A for each angle = 3.75. The distan the center of gravity of each angle from the back of the flange of the = 1.18 in and hence, $h = 1.18$ in + 3/16 in = 1.36 in, approximately. I_1 fo angle = $5.6 + 3.75 \times (1.36)^2 = 12.54$. I for each web-plate = 0.05. The m of inertia of the whole column-section, therefore, about the axis $CD =$ (2 × $(4 \times 12.54) + 0.05 = 158.21$.

Since this is the least moment of inertia the least radius of gyration wil wise be about the axis CD. The area of the cross-section of the column = 3.75, the area of the angles) + (3 × 4.5, the area of each plate) = 28.5. $158.21/28.5 = 5.55$ and r, the least radius of gyration = 2.35.

The Radius of Gyration of the Cross-Section of a Hollow Rectangular Column. As another example, let it be required to find the radius of gyration of the cross-section of a hollow rectangular cast-iron column with outside dimensions 6 by 6 in and with a shell ¼ in thick. (See figures and formulas for hollow squares and rectangles, page 335.) $A = 6^2 - (5.5)^2 = 36 - 30.25 = 5.75$ sq in. $I = (bd^3 - b_1d_1^3)/12 = [6^4 - (5.5)^4]/12 = (1\,296 - 910)/12 = 386/12 = 32.2$. $r^2 = 32.2/5.75 = 5.6$ and $r = 2.37$ in.

The radii of gyration of round-section columns and square-section columns, ranging from 2 to 20 in in diameter and of metal varying from ¼ to 2 in thick, are given in Tables II and III, see pages 348 to 351. For example: the radius of gyration of a 6 by 6-in square-section cast-iron column with a shell ¼ in thick, is, from Table III, 2.35 in.

Graphical Method of Determining the Moment of Inertia of Plane Figures *

The Moment of Inertia may be Determined Graphically as follows: Divide the shape in question, Fig. 9,† into strips parallel to *BC*. Through the centers of gravity of the strips draw indefinite lines f_1, f_2, etc. Along a line *ab* lay off lengths f_1, f_2, etc., proportional respectively to the areas f_1, f_2, etc. When the strips are narrow and of equal width, each strip may be assumed proportional to the length of the strip measured through its center of gravity. Through *a* and *b* draw lines at 45° with *ab* to determine the pole *O*, from which the rays *Od*, *Oe*, etc., are drawn. Con-

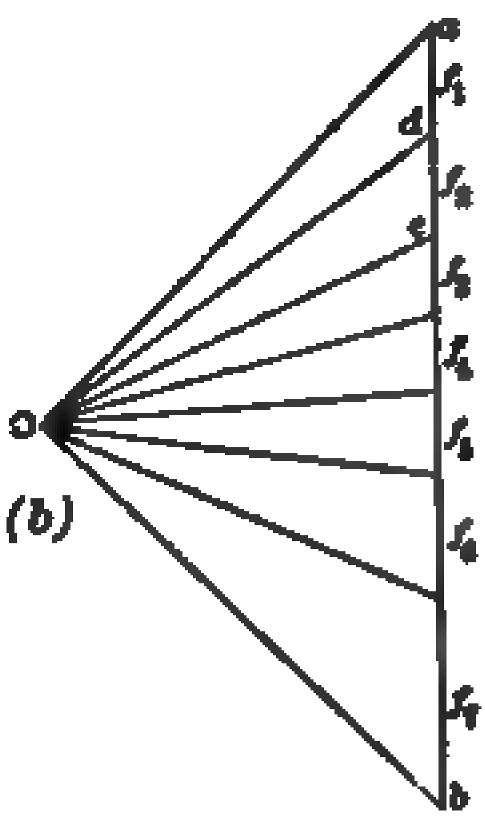


Fig. 9. Graphical Determination of Moments of Inertia†

struct the EQUILIBRIUM-POLYGON *gsf*. (See page 296.) A line *sS* parallel to *BC* will be a GRAVITY-AXIS. The MOMENT OF INERTIA about this axis is equal to the area of the given figure *ABC* multiplied by the area of the polygon *gsf*. The square root of this area *gsf* is the RADIUS OF GYRATION of the figure *ABC* with regard to the axis *sS*. A graphic method especially adopted to irregular figures is given in detail in Goodman's Mechanics Applied to Engineering. See, also, Merriman's American Civil Engineers' Handbook.

* From notes by Robins Fleming.
† Figure from Ott's Graphic Statics (Clark's Translation, London, 1876). See, also, ...'s Notes and Examples in Mechanics.

Table I.* Moments of Inertia of Rectangles

Neutral axis through center and normal to depth

Depth in inches	Widths of rectangles in inches					
	1/4	5/16	3/8	7/16	1/2	9/16
2	0.17	0.21	0.25	0.29	0.33	0.38
3	0.56	0.70	0.84	0.98	1.13	1.27
4	1.33	1.67	2.00	2.33	2.67	3.00
5	2.60	3.26	3.91	4.56	5.21	5.86
6	4.50	5.63	6.75	7.88	9.00	10.13
7	7.15	8.93	10.72	12.51	14.29	16.08
8	10.67	13.33	16.00	18.67	21.33	24.00
9	15.19	18.98	22.78	26.58	30.38	34.17
10	20.83	26.04	31.25	36.46	41.67	46.87
11	27.73	34.66	41.59	48.53	55.46	62.39
12	36.00	45.00	54.00	63.00	72.00	81.00
13	45.77	57.21	68.66	80.10	91.54	102.98
14	57.17	71.46	85.75	100.04	114.33	128.63
15	70.31	87.89	105.47	123.05	140.63	158.20
16	85.33	106.67	128.00	149.33	170.67	192.00
17	102.35	127.94	153.53	179.12	204.71	230.30
18	121.50	151.88	182.25	212.63	243.00	273.38
19	142.90	178.62	214.34	250.07	285.79	321.52
20	166.67	208.33	250.00	291.67	333.33	375.00
21	192.94	241.17	289.41	337.64	385.88	434.11
22	221.83	277.29	332.75	388.21	443.67	499.13
23	253.48	316.85	380.22	443.59	506.96	570.33
24	288.00	360.00	432.00	504.00	576.00	648.00
25	325.52	406.90	488.28	569.66	651.04	732.42
26	366.17	457.71	549.25	640.79	732.33	823.88
27	410.06	512.58	615.09	717.61	820.13	922.64
28	457.33	571.67	686.00	800.33	914.67	1029.00
29	508.10	635.13	762.16	889.18	1016.21	1143.23
30	562.50	703.13	843.75	984.38	1125.00	1265.63
32	682.67	853.33	1024.00	1194.67	1365.33	1536.00
34	818.83	1023.54	1228.25	1432.96	1637.67	1842.38
36	972.00	1215.00	1458.00	1701.00	1944.00	2187.00
38	1143.17	1428.96	1714.75	2000.54	2286.33	2572.13
40	1333.33	1666.67	2000.00	2333.33	2666.67	3000.00
42	1543.50	1929.38	2315.25	2701.13	3087.00	3472.88
44	1774.67	2218.33	2662.00	3105.67	3549.33	3993.00
46	2027.83	2534.79	3041.75	3548.71	4055.67	4562.63
48	2304.00	2880.00	3456.00	4032.00	4608.00	5184.00
50	2604.17	3255.21	3906.25	4557.29	5208.33	5859.38
52	2929.33	3661.67	4394.00	5126.33	5858.67	6591.00
54	3280.50	4100.63	4920.75	5740.88	6561.00	7381.13
56	3658.67	4573.33	5488.00	6402.67	7317.33	8232.00
58	4064.83	5081.04	6097.25	7113.46	8129.67	9145.87
60	4500.00	5625.00	6750.00	7875.00	9000.00	10125.00

* This table may be used in computing the moments of inertia of plate columns and other compound sections in which plates are used. See pages 342.

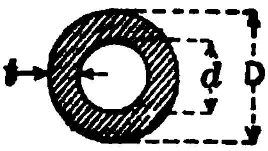
Table I* (Continued). Moments of Inertia of Rectangles

Neutral axis through center and normal to depth

Widths of rectangles in inches					
11/16	3/4	13/16	7/8	15/16	1
0.46	0.50	0.54	0.58	0.63	0.67
1.55	1.69	1.83	1.97	2.11	2.25
3.67	4.00	4.33	4.67	5.00	5.33
7.16	7.81	8.46	9.11	9.77	10.42
12.38	13.50	14.63	15.75	16.88	18.00
19.65	21.44	23.22	25.01	26.80	28.58
29.33	32.00	34.67	37.33	40.00	42.67
41.77	45.56	49.36	53.16	56.95	60.75
57.29	62.50	67.71	72.92	78.13	83.33
76.26	83.19	90.12	97.05	103.98	110.92
99.00	108.00	117.00	126.00	135.00	144.00
125.87	137.31	148.75	160.20	171.64	183.08
157.21	171.50	185.79	200.08	214.38	228.67
193.36	210.94	228.52	246.09	263.67	281.25
234.67	256.00	277.33	298.67	320.00	341.33
281.47	307.06	332.65	358.24	383.83	409.42
334.13	364.50	394.88	425.25	455.63	486.00
392.96	428.69	464.41	500.14	535.86	571.58
458.33	500.00	541.67	583.33	625.00	666.67
530.58	578.81	627.05	675.28	723.52	771.75
610.04	665.50	720.96	776.42	831.87	887.33
697.07	760.44	823.81	887.18	950.55	1013.92
792.00	864.00	936.00	1008.00	1080.00	1152.00
895.18	976.56	1057.94	1139.32	1220.70	1302.08
1006.96	1098.50	1190.04	1281.58	1373.13	1464.67
1127.67	1230.19	1332.70	1435.22	1537.73	1640.25
1257.67	1372.00	1486.33	1600.67	1715.00	1829.33
1397.29	1524.31	1651.34	1778.36	1905.39	2032.42
1546.88	1687.50	1828.13	1968.75	2109.38	2250.00
1877.33	2048.00	2218.67	2389.33	2560.00	2730.67
2251.79	2456.50	2661.21	2865.92	3070.63	3275.33
2673.00	2916.00	3159.00	3402.00	3645.00	3888.00
3143.71	3429.50	3715.29	4001.08	4286.88	4572.67
3666.67	4000.00	4333.33	4666.67	5000.00	5333.33
4244.63	4630.50	5016.38	5402.25	5788.13	6174.00
4880.33	5324.00	5767.67	6211.33	6655.00	7098.67
5576.54	6083.50	6590.46	7097.42	7604.38	8111.33
6336.00	6912.00	7488.00	8064.00	8640.00	9216.00
7161.46	7812.50	8463.54	9114.58	9765.63	10416.67
8055.67	8788.00	9520.33	10252.67	10985.00	11717.33
9021.38	9841.50	10661.63	11481.75	12301.88	13122.00
10061.33	10976.00	11890.67	12805.33	13720.00	14634.67
11178.29	12194.50	13210.71	14226.92	15243.12	16250.33
12375.00	13500.00	14625.00	15750.00	16875.00	18000.00

table may be used in computing the moments of inertia of plate girders, and other compound sections in which plates are used. See pages 341 and

Table II.* Areas and Radii of Gyration of Hollow-Round Sections



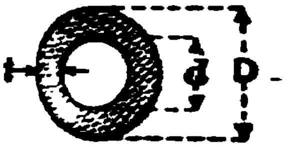
$$\text{Area} = \frac{\pi (D^2 - d^2)}{4} = 0.7854\,(D^2 - d^2)\ \text{sq in}$$

$$\text{Radius of gyration} = \frac{\sqrt{D^2 + d^2}}{4}\ \text{in}$$

Diam. D, inches	A and r	Thickness t in inches						
		1/4	5/16	3/8	1/2	5/8	3/4	7/8
2	A	1.37	1.66	......	......	......	......	......
	r	0.63	0.61	......	......	......	......	......
3	A	2.16	2.64	......	......	......	......	......
	r	0.98	0.96	......	......	......	......	......
4	A	2.95	3.62	4.27	5.50	......	......	......
	r	1.33	1.31	1.29	1.25	......	......	......
5	A	3.73	4.60	5.45	7.07	8.59	10.01	......
	r	1.68	1.66	1.64	1.60	1.56	1.53	......
6	A	4.52	5.58	6.63	8.64	10.55	12.37	14.09
	r	2.03	2.01	1.99	1.95	1.91	1.88	1.84
7	A	5.30	6.57	7.80	10.21	12.52	14.73	16.84
	r	2.39	2.37	2.35	2.30	2.27	2.23	2.19
8	A	6.09	7.55	8.98	11.78	14.48	17.08	19.59
	r	2.74	2.72	2.70	2.66	2.62	2.58	2.54
9	A	6.87	8.53	10.16	13.35	16.44	19.44	22.33
	r	3.09	3.07	3.05	3.01	2.97	2.93	2.89
10	A	7.66	9.51	11.34	14.92	18.41	21.79	25.08
	r	3.45	3.43	3.41	3.36	3.32	3.28	3.24
11	A	8.44	10.49	12.52	16.49	20.37	24.15	27.83
	r	3.80	3.78	3.76	3.72	3.67	3.63	3.59
12	A	9.23	11.47	13.70	18.06	22.33	26.51	30.58
	r	4.16	4.13	4.11	4.07	4.03	3.99	3.95
13	A	10.01	12.46	14.87	19.63	24.30	28.86	33.33
	r	4.51	4.49	4.47	4.42	4.38	4.34	4.30
14	A	10.80	13.44	16.05	21.21	26.26	31.22	36.08
	r	4.86	4.84	4.82	4.78	4.73	4.69	4.65
15	A	11.58	14.42	17.23	22.78	28.23	33.58	38.83
	r	5.22	5.19	5.17	5.13	5.09	5.05	5.00
16	A	12.37	15.40	18.41	24.35	30.19	35.93	41.58
	r	5.57	5.55	5.53	5.48	5.44	5.40	5.36
17	A	13.16	16.38	19.59	25.92	32.15	38.29	44.33
	r	5.92	5.90	5.88	5.84	5.79	5.75	5.71
18	A	13.94	17.36	20.76	27.49	34.12	40.64	47.07
	r	6.28	6.25	6.23	6.19	6.15	6.10	6.06
19	A	14.73	18.35	21.94	29.06	36.08	43.00	49.82
	r	6.63	6.61	6.59	6.54	6.50	6.46	6.42
20	A	15.51	19.33	23.12	30.63	38.04	45.36	52.57
	r	6.98	6.96	6.94	6.90	6.85	6.81	6.77

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

F[1] (Continued). Areas and Radii of Gyration of Hollow-Round



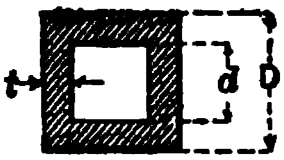
$$\text{Area} = \frac{\pi (D^2 - d^2)}{4} = 0.7854\,(D^2 - d^2) \text{ sq in}$$

$$\text{Radius of gyration} = \frac{\sqrt{D^2 + d^2}}{4} \text{ in}$$

D, inches	A and r	Thickness t in inches						
		1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞
2	A	......	......	......	......	......	......	......
	r	......	......	......	......	......	......	......
3	A	......	......	......	......	......	......	......
	r	......	......	......	......	......	......	......
4	A	......	......	......	......	......	......	......
	r	......	......	......	......	......	......	......
5	A	......	......	......	......	......	......	......
	r	......	......	......	......	......	......	......
6	A	......	......	......	......	......	......	......
	r	......	......	......	......	......	......	......
[illegible]	A	20.76	22.58	......	......	......	......	......
	r	2.12	2.08	......	......	......	......	......
[illegible]	A	24.30	26.51	28.62	30.63	......	......	......
	r	2.46	2.43	2.39	2.36	......	......	......
[illegible]	A	27.83	30.43	32.94	35.34	37.65	39.86	......
	r	2.81	2.78	2.74	2.70	2.67	2.64	......
[illegible]	A	31.37	34.36	37.26	40.06	42.76	45.36	47.86
	r	3.16	3.13	3.09	3.05	3.02	2.98	2.95
[illegible]	A	34.90	38.29	41.58	44.77	47.86	50.85	53.75
	r	3.51	3.48	3.44	3.40	3.36	3.33	3.29
[illegible]	A	38.44	41.22	45.90	49.48	52.97	56.35	59.64
	r	3.87	3.83	3.79	3.75	3.71	3.68	3.64
[illegible]	A	41.97	46.14	50.22	54.19	58.07	61.85	65.53
	r	4.22	4.18	4.14	4.10	4.06	4.03	5.99
[illegible]	A	45.50	50.07	54.54	58.91	63.18	67.35	71.42
	r	4.57	4.53	4.49	4.45	4.41	4.38	4.34
[illegible]	A	49.04	54.00	58.86	63.62	68.28	72.85	77.31
	r	4.92	4.88	4.84	4.80	4.76	4.73	4.69
[illegible]	A	52.57	57.92	63.18	68.33	73.39	78.34	83.20
	r	5.27	5.23	5.19	5.15	5.11	5.08	5.04
[illegible]	A	56.11	61.85	67.50	73.04	78.49	83.84	89.09
	r	5.63	5.59	5.55	5.51	5.47	5.43	5.39
[illegible]	A	59.64	65.78	71.82	77.75	83.60	89.34	94.98
	r	5.98	5.94	5.90	5.86	5.82	5.78	5.74
[illegible]	A	63.18	69.70	76.13	82.47	88.70	94.84	100.87
	r	6.33	6.29	6.25	6.21	6.17	6.13	6.09
[illegible]	A	66.71	73.63	80.45	87.18	93.81	100.33	106.77
	r	6.69	6.64	6.60	6.56	6.52	6.48	6.44

[1] From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa

Table III.* Areas and Radii of Gyration of Hollow-Square Sections



Area = $(D^2 - d^2)$ sq in

Radius of gyration = $\sqrt{\dfrac{D^2 + d^2}{12}}$ in

Side D, inches	A and r	Thickness t in inches						
		¼	5/16	⅜	½	⅝	¾	⅞
2	A	1.75	2.11	......	......	......	......	......
	r	0.72	0.70	......	......	......	......	......
3	A	2.75	3.36	......	......	......	......	......
	r	1.13	1.10	......	......	......	......	......
4	A	3.75	4.61	5.44	7.00	......	......	......
	r	1.53	1.51	1.49	1.44	......	......	......
5	A	4.75	5.86	6.94	9.00	10.94	12.75	......
	r	1.94	1.92	1.89	1.85	1.80	1.76	......
6	A	5.75	7.11	8.44	11.00	13.44	15.75	17.94
	r	2.35	2.33	2.30	2.25	2.21	2.17	2.12
7	A	6.75	8.36	9.94	13.00	15.94	18.75	21.44
	r	2.76	2.73	2.71	2.66	2.62	2.57	2.53
8	A	7.75	9.61	11.44	15.00	18.44	21.75	24.94
	r	3.17	3.14	3.12	3.07	3.02	2.98	2.93
9	A	8.75	10.86	12.94	17.00	20.94	24.75	28.44
	r	3.57	3.55	3.53	3.48	3.43	3.38	3.34
10	A	9.75	12.11	14.44	19.00	23.44	27.75	31.94
	r	3.98	3.96	3.93	3.88	3.84	3.79	3.74
11	A	10.75	13.36	15.94	21.00	25.94	30.75	35.44
	r	4.39	4.37	4.34	4.29	4.24	4.20	4.15
12	A	11.75	14.61	17.44	23.00	28.44	33.75	38.94
	r	4.80	4.77	4.75	4.70	4.65	4.60	4.56
13	A	12.75	15.86	18.94	25.00	30.94	36.75	42.44
	r	5.21	5.18	5.16	5.11	5.06	5.01	4.96
14	A	13.75	17.11	20.44	27.00	33.44	39.75	45.94
	r	5.61	5.59	5.56	5.51	5.47	5.42	5.37
15	A	14.75	18.36	21.94	29.00	35.94	42.75	49.44
	r	6.02	6.00	5.97	5.92	5.87	5.83	5.78
16	A	15.75	19.61	23.44	31.00	38.44	45.75	52.94
	r	6.43	6.41	6.38	6.33	6.28	6.23	6.19
17	A	16.75	20.86	24.94	33.00	40.94	48.75	56.44
	r	6.84	6.81	6.79	6.74	6.69	6.64	6.59
18	A	17.75	22.11	26.44	35.00	43.44	51.75	59.94
	r	7.25	7.22	7.20	7.15	7.10	7.05	7.00
19	A	18.75	23.36	27.94	37.00	45.94	54.75	63.44
	r	7.66	7.63	7.61	7.56	7.51	7.46	7.41
20	A	19.75	24.61	29.44	39.00	48.44	57.75	66.94
	r	8.06	8.04	8.01	7.96	7.91	7.87	7.82

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa

...le II* (Continued). Areas and Radii of Gyration of Hollow-Squa...

Area = $(D^2 - d^2)$ sq in

Radius of gyration = $\sqrt{\dfrac{D^2 + d^2}{12}}$ in

Side D, inches	A and r	Thickness t in inches						
		1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞
2	A	.....	.....	......	......	......	......	......
	r	.....	.....	......	......	......	......	......
3	A	.....	.....	......	......	......	......	......
	r	.....	.....	......	......	......	......	......
4	A	.....	.....	......	......	......	......	......
	r	.....	.....	......	......	......	......	......
5	A	.....	.....	......	......	......	......	......
	r	.....	.....	......	......	......	......	......
6	A	.....	.....	......	......	......	......	......
	r	.....	.....	......	......	......	......	......
7	A	26.44	28.75	......	......	......	......	......
	r	2.44	2.40	......	......	......	......	......
8	A	30.94	33.75	36.44	39.00	......	......	......
	r	2.84	2.80	2.76	2.72	......	......	......
9	A	35.44	38.75	41.94	45.00	47.94	50.75	......
	r	3.25	3.20	3.16	3.12	3.08	3.05	......
10	A	39.94	43.75	47.44	51.00	54.44	57.75	60.94
	r	3.65	3.61	3.57	3.52	3.48	3.44	3.40
11	A	44.44	48.75	52.94	57.00	60.94	64.75	68.44
	r	4.06	4.01	3.97	3.93	3.88	3.84	3.80
12	A	48.94	53.75	58.44	63.00	67.44	71.75	75.94
	r	4.46	4.42	4.37	4.33	4.29	4.25	4.20
13	A	53.44	58.75	63.94	69.00	73.94	78.75	83.44
	r	4.87	4.82	4.78	4.74	4.69	4.65	4.61
14	A	57.94	63.75	69.44	75.00	80.44	85.75	90.94
	r	5.28	5.23	5.18	5.14	5.10	5.05	5.01
15	A	62.44	68.75	74.94	81.00	86.94	92.75	98.44
	r	5.68	5.64	5.59	5.55	5.50	5.46	5.41
16	A	66.94	73.75	80.44	87.00	93.44	99.75	105.94
	r	6.09	6.04	6.00	5.95	5.91	5.86	5.82
17	A	71.44	78.75	85.94	93.00	99.94	106.75	113.44
	r	6.50	6.45	6.40	6.36	6.31	6.27	6.23
18	A	75.94	83.75	91.44	99.00	106.44	113.75	120.94
	r	6.90	6.86	6.82	6.76	6.72	6.67	6.63
19	A	80.44	88.75	96.94	105.00	112.94	120.75	128.44
	r	7.31	7.26	7.22	7.17	7.12	7.08	7.03
20	A	84.94	93.75	102.44	111.00	119.44	127.75	135.94
	r	7.72	7.67	7.62	7.58	7.53	7.49	7.44

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, P...

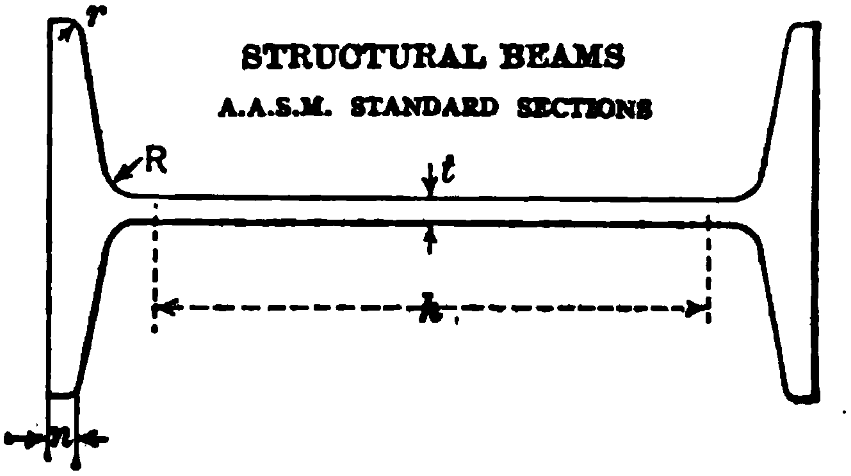
7. Dimensions, Moments of Inertia, Radii of Gyration and Sec Moduli of Standard Structural Shapes

Explanation of Tables. As in using structural-steel shapes the ch practically confined to such shapes as are rolled by the mills, it is essent have at hand the dimensions and properties of those shapes in order to late the necessary sizes to meet special requirements for strength and pr conditions of economy and framing. Since 1890 great changes have been both in the materials and in the shapes of the standard sections. The r mills which manufacture the most complete assortment of structural sha those of the Carnegie Steel Company, the Cambria Steel Company, the & Laughlin Steel Company and the Bethlehem Steel Company. In gener products of these mills, especially beams and channels, are respectively s in shape. This is particularly true of the shapes rolled by the first three companies named.

The standard steel beams and channels considered in the following pa rolled by all of these mills, with the exception of those of the Bethlehen Company. With a few exceptions the following tables of properties of ard structural shapes have been adopted by permission from the l Companion of the Carnegie Steel Company. It may be well to stat the tables of properties for the various structural shapes, published i companies named above, do not agree exactly, even for the same w but the differences are not of practical importance. The tables Cambria Steel Company and of the Carnegie Steel Company for bear channels agree more closely. As angles are very extensively used great many purposes, the properties are given for all sizes rolled and table showing from which mills the different sizes may be obtained. ally it will generally be advantageous to use a size that is rolled by mills.

Tables XV, XVI and XVII will be found very convenient when com the strength of struts formed of pairs of angles, and Table XVIII whe puting the same for pairs of channels.

Standard Steel Beams and Channels.* **Common Dimension**



n = minimum web = t

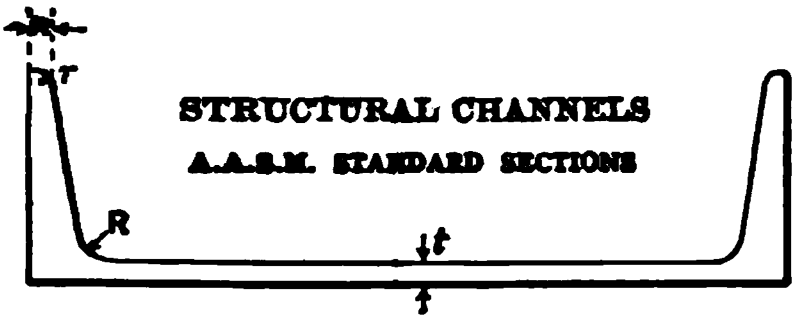
R = minimum web + 0.10″

r = 5/10 minimum web

h = distance between flange-fillets

Slope of flange, 1 : 6 = $16\frac{2}{3}\%$ = 9° 27′ 44″

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.



n = minimum web = t

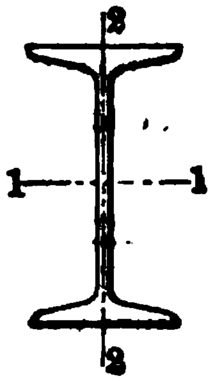
R = minimum web + 0.10″

r = $\frac{9}{10}$ minimum web

Slope of flange, 1 : 6 = $16\frac{2}{3}\%$ = 9° 27′ 44″

ensions for Structural Beams are those adopted by the Association of ican Steel Manufacturers and apply to all Structural Beams, except ican Standard Sections B 1, B 2 and B 3, also Sections B 24 and B 81. e dimensions of the Supplementary Beams, B 61 to B 68, inclusive, t be readily reduced to formulas. Slope of flange is 1 : 11 = 5° 11′ 40″. ensions for Structural Channels are those adopted by the Association erican Steel Manufacturers and apply to all Structural Channels, except s C 10, the 13-in sizes, which are Car Building Channels.

Table IV.* Properties of I-Beam Sections

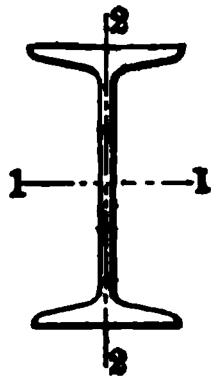


Section-index	Depth of beam	Weight per foot	Area of section	Width of flange	Thickness of web	Axis 1-1			Axis 2-	
						I	r	I/c	I	r
	in	lb	sq in	in	in	in^4	in	in^3	in^4	in
B 61	27	90.0	26.34	9.000	0.524	2958.3	10.60	219.1	75.3	1.69
B 18	24	115.0	33.67	7.987	0.737	2940.5	9.35	245.0	82.8	1.57
		110.0	32.18	7.925	0.675	2869.1	9.44	239.1	80.6	1.58
		105.9	30.98	7.875	0.625	2811.5	9.53	234.3	78.9	1.60
B 1	24	100.0	29.25	7.247	0.747	2371.8	9.05	197.6	48.4	1.[illegible]
		95.0	27.79	7.186	0.686	2301.5	9.08	191.8	47.0	1.30
		90.0	26.30	7.124	0.624	2230.1	9.21	185.8	45.5	1.32
		85.0	24.84	7.063	0.563	2159.8	9.33	180.0	44.2	1.3[illegible]
		79.9	23.33	7.000	0.500	2087.2	9.46	173.9	42.9	1.36
B 62	24	74.2	21.70	9.000	0.476	1950.1	9.48	162.5	61.2	1.68
B 63	21	60.4	17.68	8.250	0.428	1235.5	8.36	117.7	43.5	1.57
B 2	20	100.0	29.20	7.273	0.873	1648.3	7.51	164.8	52.4	1.34
		95.0	27.74	7.200	0.800	1599.7	7.59	160.0	50.5	1.3[illegible]
		90.0	26.26	7.126	0.726	1550.3	7.68	155.0	48.7	1.3[illegible]
		85.0	24.80	7.053	0.653	1501.7	7.78	150.2	47.0	1.3[illegible]
		81.4	23.74	7.000	0.600	1466.3	7.86	146.6	45.8	1.3[illegible]
B 3	20	75.0	21.90	6.391	0.641	1263.5	7.60	126.3	30.1	1.1[illegible]
		70.0	20.42	6.317	0.567	1214.2	7.71	121.4	28.9	1.1[illegible]
		65.4	19.08	6.250	0.500	1169.5	7.83	116.9	27.9	1.[illegible]
B 19	18	90.0	26.29	7.236	0.796	1256.5	6.91	139.6	51.9	1.4[illegible]
		85.0	24.81	7.154	0.714	1216.6	7.00	135.2	49.8	1.4[illegible]
		80.0	23.34	7.072	0.632	1176.8	7.10	130.8	47.9	1.4[illegible]
		75.6	22.04	7.000	0.560	1141.8	7.20	126.9	46.3	1.4[illegible]
B 4	18	70.0	20.46	6.251	0.711	917.5	6.70	101.9	24.5	1.[illegible]
		65.0	18.98	6.169	0.629	877.7	6.80	97.5	23.4	1.[illegible]
		60.0	17.50	6.087	0.547	837.8	6.92	93.1	22.3	1.[illegible]
		54.7	15.94	6.000	0.460	795.5	7.07	88.4	21.2	1.[illegible]
B 64	18	48.2	14.09	7.500	0.380	737.1	7.23	81.9	30.0	1.[illegible]
B 6	15	75.0	21.85	6.278	0.868	687.2	5.61	91.6	30.6	1.[illegible]
		70.0	20.38	6.180	0.770	659.6	5.69	87.9	28.8	1.[illegible]
		65.0	18.91	6.082	0.672	632.1	5.78	84.3	27.2	1.[illegible]
		60.8	17.68	6.000	0.590	609.0	5.87	81.2	26.0	1.[illegible]
B 7	15	55.0	16.06	5.738	0.648	508.7	5.63	67.8	17.0	1.[illegible]
		50.0	14.59	5.640	0.550	481.1	5.74	64.2	16.0	1[illegible]
		45.0	13.12	5.542	0.452	453.6	5.88	60.5	15.0	1[illegible]
		42.9	12.49	5.500	0.410	441.8	5.95	58.9	14.6	1[illegible]
B 65	15	37.3	10.91	6.750	0.332	405.5	6.10	54.1	19.9	1[illegible]

NOTE. The exponential figures used with I and I/c denote the mathem[illegible] mensions" of these qualities, that is, the number of times the linear unit a[illegible] factor in the quantities.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, P[illegible]

Table IV* (Continued). Properties of I-Beam Section

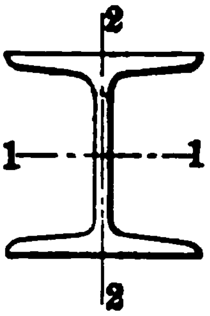


[illegible]	Depth of beam	Weight per foot	Area of section	Width of flange	Thickness of web	Axis 1-1 I	Axis 1-1 r	Axis 1-1 I/c	Axis 2-2 I	Axis 2-2 r	Axis 2-2 I/c
	in	lb	sq in	in	in	in⁴	in	in³	in⁴	in	in³
[illegible]	12	55.0	16.04	5.600	0.810	319.3	4.46	53.2	17.3	1.04	6.2
		50.0	14.57	5.477	0.687	301.6	4.55	50.3	16.0	1.05	5.8
		45.0	13.10	5.355	0.565	284.1	4.66	47.3	14.8	1.06	5.5
		40.8	11.84	5.250	0.460	268.9	4.77	44.8	13.8	1.08	5.3
[illegible]	12	35.0	10.20	5.078	0.428	227.0	4.72	37.8	10.0	0.99	3.9
		31.8	9.26	5.000	0.350	215.8	4.83	36.0	9.5	1.01	3.8
[illegible]	12	27.9	8.15	6.000	0.284	199.4	4.95	33.2	12.6	1.24	4.2
[illegible]	10	40.0	11.69	5.091	0.741	158.0	3.68	31.6	9.4	0.90	3.7
		35.0	10.22	4.944	0.594	145.8	3.78	29.2	8.5	0.91	3.4
		30.0	8.75	4.797	0.447	133.5	3.91	26.7	7.6	0.93	3.2
		25.4	7.38	4.660	0.310	122.1	4.07	24.4	6.9	0.97	3.0
[illegible]	10	22.4	6.54	5.500	0.252	113.6	4.17	22.7	9.0	1.17	3.3
[illegible]	9	35.0	10.22	4.764	0.724	111.3	3.30	24.7	7.3	0.84	3.0
		30.0	8.76	4.601	0.561	101.4	3.40	22.5	6.4	0.85	2.8
		25.0	7.28	4.437	0.397	91.4	3.54	20.3	5.6	0.88	2.5
		21.8	6.32	4.330	0.290	84.9	3.67	18.9	5.2	0.90	2.4
[illegible]	8	25.5	7.43	4.262	0.532	68.1	3.03	17.0	4.7	0.80	2.2
		23.0	6.71	4.171	0.441	64.2	3.09	16.0	4.4	0.81	2.1
		20.5	5.97	4.079	0.349	60.2	3.18	15.1	4.0	0.82	2.0
		18.4	5.34	4.000	0.270	56.9	3.26	14.2	3.8	0.84	1.9
[illegible]	8	17.5	5.13	5.000	0.220	58.4	3.38	14.6	6.2	1.10	2.5
[illegible]	7	20.0	5.83	3.860	0.450	41.9	2.68	12.0	3.1	0.74	1.6
		17.5	5.09	3.755	0.345	38.9	2.77	11.1	2.9	0.76	1.6
		15.3	4.43	3.660	0.250	36.2	2.86	10.4	2.7	0.78	1.5
[illegible]	6	17.25	5.02	3.565	0.465	26.0	2.28	8.7	2.3	0.68	1.3
		14.75	4.29	3.443	0.343	23.8	2.36	7.9	2.1	0.69	1.2
		12.5	3.61	3.330	0.230	21.8	2.46	7.3	1.8	0.72	1.1
[illegible]	5	14.75	4.29	3.284	0.494	15.0	1.87	6.0	1.7	0.63	1.0
		12.25	3.56	3.137	0.347	13.5	1.95	5.4	1.4	0.63	0.91
		10.0	2.87	3.000	0.210	12.1	2.05	4.8	1.2	0.65	0.82
[illegible]	4	10.5	3.05	2.870	0.400	7.1	1.52	3.5	1.0	0.57	0.70
		9.5	2.76	2.796	0.326	6.7	1.56	3.3	0.91	0.58	0.65
		8.5	2.46	2.723	0.253	6.3	1.60	3.2	0.83	0.58	0.61
		7.7	2.21	2.660	0.190	6.0	1.64	3.0	0.77	0.59	0.58
[illegible]	3	7.5	2.17	2.509	0.349	2.9	1.15	1.9	0.59	0.52	0.47
		6.5	1.88	2.411	0.251	2.7	1.19	1.8	0.51	0.52	0.43
		5.7	1.64	2.330	0.170	2.5	1.23	1.7	0.46	0.53	0.40

[illegible] "Note" with table on preceding page.

*From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table V.[*] Properties of H-Beam Sections



These may be employed as columns, using the axis 2-2

Section-index	Depth of beam	Weight per foot	Area of section	Width of flange	Thickness of web	Axis 1-1			Axis 2-2	
						I	r	I/c	I	r
	in	lb	sq in	in	in	in^4	in	in^3	in^4	in
H 4	8	34.3	10.01	8.0	0.375	115.4	3.40	28.9	35.1	1.87
H 3	6	24.1	7.01	6.0	0.313	45.1	2.54	15.0	14.7	1.45
H 2	5	18.9	5.47	5.0	0.313	23.8	2.08	9.5	7.9	1.20
H 1	4	13.8	4.00	4.0	0.313	10.7	1.63	5.3	3.6	0.95

See " Note " with Table IV, page 354.

[*] From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table VI.* Properties of Bethlehem I-Beam Sections

Depth of beam	Weight per foot	Area of section	Thickness of web	Width of flange	Increase of web and flange for each lb increase of weight	Neutral axis perpendicular to web at center			Neutral axis coincident with center line of web	
						I	r	I/c	I	r
in	lb	sq in	in	in	in	in^4	in	in^3	in^4	in
30	120.0	35.30	0.540	10.500	0.010	5239.6	12.18	349.3	165.0	2.16
28	105.0	30.88	0.500	10.000	0.011	4014.1	11.40	286.7	131.5	2.06
26	90.0	26.49	0.460	9.500	0.011	2977.2	10.60	229.0	101.2	1.95
24	84.0	24.80	0.460	9.250	0.012	2381.9	9.80	198.5	91.1	1.92
24	83.0	24.59	0.520	9.130	0.012	2240.9	9.55	186.7	78.0	1.78
24	73.0	21.47	0.390	9.000	0.012	2091.0	9.87	174.3	74.4	1.86
20	82.0	24.17	0.570	8.890	0.015	1559.8	8.03	156.0	79.9	1.82
20	72.0	21.37	0.430	8.750	0.015	1466.5	8.28	146.7	75.9	1.88
20	69.0	20.26	0.520	8.145	0.015	1268.9	7.91	126.9	51.2	1.59
20	64.0	18.86	0.450	8.075	0.015	1222.1	8.05	122.2	49.8	1.62
20	59.0	17.36	0.375	8.000	0.015	1172.2	8.22	117.2	48.3	1.66
18	59.0	17.40	0.495	7.675	0.016	883.3	7.12	98.1	39.1	1.50
18	54.0	15.87	0.410	7.590	0.016	842.0	7.28	93.6	37.7	1.54
18	52.0	15.24	0.375	7.555	0.016	825.0	7.36	91.7	37.1	1.56
18	48.5	14.25	0.320	7.500	0.016	798.3	7.48	88.7	36.2	1.59
15	71.0	20.95	0.520	7.500	0.020	796.2	6.16	106.2	61.3	1.71
15	64.0	18.81	0.605	7.195	0.020	664.9	5.95	88.6	41.9	1.49
15	54.0	15.88	0.410	7.000	0.020	610.0	6.20	81.3	38.3	1.55
15	46.0	13.52	0.440	6.810	0.020	484.8	5.99	64.6	25.2	1.36
15	41.0	12.02	0.340	6.710	0.020	456.7	6.16	60.9	24.0	1.41
15	38.0	11.27	0.290	6.660	0.020	442.6	6.27	59.0	23.4	1.44
12	36.0	10.61	0.310	6.300	0.025	269.2	5.04	44.9	21.3	1.42
12	32.0	9.44	0.335	6.205	0.025	228.5	4.92	38.1	16.0	1.30
12	28.5	8.42	0.290	6.120	0.025	216.2	5.07	36.0	15.3	1.35
10	28.5	8.34	0.390	5.990	0.029	134.6	4.02	26.9	12.1	1.21
10	23.5	6.94	0.250	5.850	0.029	122.9	4.21	24.6	11.2	1.27
9	24.0	7.04	0.365	5.555	0.033	92.1	3.62	20.5	8.8	1.12
9	20.0	6.01	0.250	5.440	0.033	85.1	3.76	18.9	8.2	1.17
8	19.5	5.78	0.325	5.325	0.037	60.6	3.24	15.1	6.7	1.08
8	17.5	5.18	0.250	5.250	0.037	57.4	3.33	14.3	6.4	1.11

* See "Note" with Table IV, page 354.

Adapted from Catalogue of Structural Shapes, 1911 Edition, Bethlehem Steel Company, Bethlehem, Pa. See nine Additional Sections, for 24-in, 22-in, and 18-in beams, Pamphlet S-10, published March, 1, 1921, by this Company.

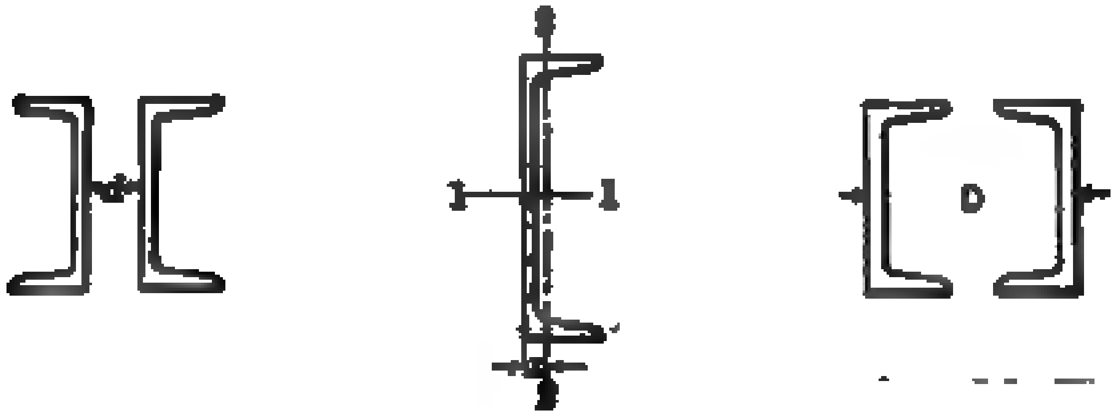
Table VII.* Properties of Bethlehem Girder-Beam Sections

Depth of beam	Weight per foot	Area of section	Thickness of web	Width of flange	Increase of web and flange for each pound increase of weight	Neutral axis perpendicular to web at center			Neutral axis coincident with center line of web	
						I	r	I/c	I	r
in	lb	sq in	in	in	in	in⁴	in	in³	in⁴	in
30	200.0	58.71	0.750	15.00	0.010	9150.6	12.48	610.0	630.2	3.
30	180.0	53.00	0.690	13.00	0.010	8194.5	12.43	546.3	433.3	2.
28	180.0	52.86	0.690	14.35	0.011	7264.7	11.72	518.9	533.3	3.
28	165.0	48.47	0.660	12.50	0.011	6562.7	11.64	468.8	371.9	2.
26	160.0	46.91	0.630	13.60	0.011	5620.8	10.95	432.4	435.7	3.
26	150.0	43.94	0.630	12.00	0.011	5153.9	10.83	396.5	314.6	2.
24	140.0	41.16	0.600	13.00	0.012	4201.4	10.10	350.1	346.9	2.
24	120.0	35.38	0.530	12.00	0.012	3607.3	10.10	300.6	249.4	2.
20	140.0	41.19	0.640	12.50	0.015	2934.7	8.44	293.5	348.9	2.
20	112.0	32.81	0.550	12.00	0.015	2342.1	8.45	234.2	239.3	2.
18	92.0	27.12	0.480	11.50	0.016	1591.4	7.66	176.8	182.6	2.
15	140.0	41.27	0.800	11.75	0.020	1592.7	6.21	212.4	331.0	2.
15	104.0	30.50	0.600	11.25	0.020	1220.1	6.32	162.7	213.0	2.
15	73.0	21.49	0.430	10.50	0.020	883.4	6.41	117.8	123.2	2.
12	70.0	20.58	0.460	10.00	0.025	538.8	5.12	89.8	114.7	2
12	55.0	16.18	0.370	9.75	0.025	432.0	5.17	72.0	81.1	2
10	44.0	12.95	0.310	9.00	0.030	244.2	4.34	48.8	57.3	2
9	38.0	11.22	0.300	8.50	0.033	170.9	3.90	38.0	44.1	1
8	32.5	9.54	0.290	8.00	0.037	114.4	3.46	28.6	32.9	1

See "Note" with Table IV, page 354.

* Adapted from Catalogue of Structural Shapes, 1911 Edition, Bethlehem Company, Bethlehem, Pa.

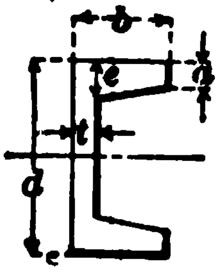
Table VIII.* Properties of Channel-Sections



Depth of chan-nel	Weight per foot	Area of sec-tion	Width of flange	T…	Axis 2–2			x	d†
					I	r	I/c		
in	lb	sq in	in		in⁴	in	in³	in	
15	55.0	16.11	3.814	0	12.1	0.87	4.1	0.82	8.53
	50.0	14.64	3.716	0	11.2	0.87	3.8	0.80	8.71
	45.0	13.17	3.618	0	10.3	0.88	3.6	0.79	8.92
	40.0	11.70	3.520	0	9.3	0.89	3.4	0.78	9.15
	35.0	10.23	3.422	0	8.4	0.91	3.2	0.79	9.43
	33.9	9.90	3.400	0	8.2	0.91	3.2	0.79	9.50
12	40.0	11.73	3.415	0	6.6	0.75	2.5	0.72	6.60
	35.0	10.26	3.292	0	5.9	0.76	2.3	0.69	6.81
	30.0	8.79	3.170	0	5.2	0.77	2.1	0.68	7.07
	25.0	7.32	3.047	0	4.5	0.79	1.9	0.68	7.36
	20.7	6.03	2.940	0	3.9	0.81	1.7	0.70	7.67
10	35.0	10.27	3.180	0	4.6	0.67	1.9	0.69	5.17
	30.0	8.80	3.033	0	4.0	0.67	1.7	0.65	5.40
	25.0	7.33	2.886	0	3.4	0.68	1.5	0.62	5.67
	20.0	5.86	2.739	0	2.8	0.70	1.3	0.61	5.97
	15.3	4.47	2.600	0	2.3	0.72	1.2	0.64	6.33
9	25.0	7.33	2.812	0	3.0	0.64	1.4	0.61	4.84
	20.0	5.86	2.648	0	2.4	0.65	1.2	0.59	5.12
	15.0	4.39	2.485	0	1.9	0.67	1.0	0.59	5.49
	13.4	3.89	2.430	0	1.8	0.67	0.97	0.61	5.63
8	21.25	6.23	2.619	0	2.2	0.60	1.1	0.59	4.23
	18.75	5.49	2.527	0	2.0	0.62	1.0	0.57	4.38
	16.25	4.76	2.435	0	1.8	0.61	0.94	0.56	4.54
	13.75	4.02	2.343	0	1.5	0.62	0.86	0.56	4.72
	11.5	3.36	2.260	0	1.3	0.63	0.79	0.58	4.94
7	19.75	5.79	2.509	0	1.8	0.56	0.96	0.58	3.48
	17.25	5.05	2.404	0	1.6	0.56	0.86	0.55	3.64
	14.75	4.32	2.299	0	1.4	0.57	0.79	0.53	3.80
	12.25	3.58	2.194	0	1.2	0.58	0.71	0.53	3.99
	9.8	2.85	2.090	0	0.98	0.59	0.63	0.55	4.22
6	15.5	4.54	2.279	0	1.3	0.53	0.73	0.55	2.91
	13.0	3.81	2.157	0	1.1	0.53	0.65	0.52	3.09
	10.5	3.07	2.034	0	0.87	0.53	0.57	0.50	3.28
	8.2	2.39	1.920	0	0.70	0.54	0.50	0.52	3.52
5	11.5	3.36	2.032	0	0.82	0.49	0.54	0.51	2.34
	9.0	2.63	1.885	0	0.64	0.49	0.45	0.48	2.56
	6.7	1.95	1.750	0	0.48	0.50	0.38	0.49	2.79
4	7.25	2.12	1.720	0	0.44	0.46	0.35	0.46	1.85
	6.25	1.82	1.647	0	0.38	0.45	0.32	0.46	1.96
	5.4	1.56	1.580	0	0.32	0.45	0.29	0.46	2.06
3	6.0	1.75	1.596	0	0.31	0.42	0.27	0.46	1.07
	5.0	1.46	1.498	0	0.25	0.41	0.24	0.44	1.19
	4.1	1.19	1.410	0	0.20	0.41	0.21	0.44	1.31

*[Ar]ranged from Pocket Companion, Carnegie Steel Company, Pittsburgh, … "Note" with Table IV, page 354. † These values make r the same for both …

Table IX.* Dimensions of Sections and Weights of Small Grooved Steel Channels



Section-index	Size of section, in	Width of flange, in	Thickness of web, in	Weight per foot, lb
C-164	2⅛	13/16	¼	2.55
C-165	2⅛	¾	3/16	2.09
C-166	2⅛	11/16	⅛	1.63
C-183	2	⅝	¼	2.11
C-184	2	9/16	3/16	1.68
C-185	2	½	⅛	1.26
C-190	1¾	11/16	3/16	1.71
C-191	1¾	⅝	⅛	1.33
C-193	1¾	17/32	5/32	1.33
C-195	1¾	⅜	⅛	0.96
C-197	1½	⅝	3/16	1.47
C-199	1¼	½	⅛	0.93
C-200	1⅛	9/16	3/16	1.12
C-203	1	½	⅛	0.83
C-207	1	25/64	⅛	0.71
C-213	⅞	7/16	⅛	0.66
C-217	⅞	⅜	⅛	0.58
C-219	¾	⅜	⅛	0.54
C-221	¾	11/32	3/32	0.40
C-223	⅝	5/16	⅛	0.43

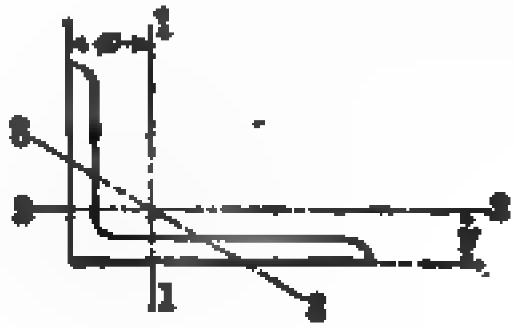
* Rolled by the Jones & Laughlin Steel Company, Pittsburgh, Pa.

Table X. Sizes and Makes of Rolled Steel Angles

[illegible] following table has been compiled to show angles of various sizes rolled by [illegible] companies. The word "all" indicates that angles of the sizes mentioned are [illegible]d by all the companies included in the list. The abbreviations refer to the [illegible] companies: Cam., Cambria Steel Company; Car., Carnegie Steel Com[illegible]; J. & L., Jones & Laughlin Steel Company.

Angles with unequal legs		Angles with equal legs	
[illegible] in inches	Companies	Sizes in inches	Companies
8 ×6	Cam. and Car.	8 ×8	All
8 ×3½	Car.	6 ×6	All
7 ×3½	All	5 ×5	All
6 ×4	All	4½×4½	Cam.
6 ×3½	All	4 ×4	All
5 ×4	All	3½×3½	All
5 ×3½	All	3¼×3¼	J. & L.
5 ×3	All	3 ×3	All
4½×3	Car. and J. & L.	2¾×2¾	Cam. and J. & L.
4 ×3½	All	2½×2½	All
4 ×3	All	2¼×2¼	Cam. and J. & L.
3½×3	All	2 ×2	All
3½×2½	All	1¾×1¾	All
3½×2	J. & L.	1½×1½	All
3½×1½	J. & L.	1⅜×1⅜	J. & L.
3 ×2½	All	1¼×1¼	All
3 ×2	All	1³⁄₁₆×1³⁄₁₆	J. & L.
2½×2	All	1 ×1	All
2½×1¾	J. & L.	¾× ¾	J. & L.
2½×1½	Car. and J. & L.		
2½×1¼	Cam.		
2¼×1⅜	J. & L.		
2¼×1½	Car. and J. & L.		
2 ×1½	Car. and J. & L.		
2 ×1⅜	J. & L.		
2 ×1¼	Car.		
2 ×1	J. & L.		
[illegible]×1½	J. & L.		
[illegible]×1¼	Car.		
[illegible]×1⅛	J. & L.		
[illegible]×1¼	Car.		
[illegible]×1	J. & L.		
[illegible]× ⅞	J. & L.		
[illegible] ×¹³⁄₁₆	J. & L.		
[illegible] × ⅝	J. & L.		

Table XI.* Properties of Angle-Sections.* Unequal Legs.

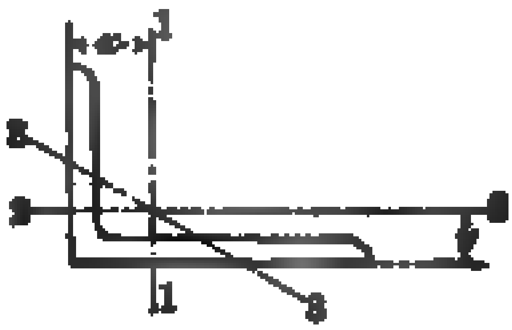


			Axis 1–1				Axis 2–2			
			I	r	I/c	x	I	r	I/c	y
			in^4	in	in^3	in	in^4	in	in^3	in
8X6 X1	44.2	13.00	80.8	2.49	15.1	2.65	38.8	1.73	8.9	1.65
8X6 X15/16	41.7	12.25	76.6	2.50	14.3	2.63	36.8	1.73	8.4	1.63
8X6 X7/8	39.1	11.48	72.3	2.51	13.4	2.61	34.9	1.74	7.9	1.61
8X6 X13/16	36.5	10.72	67.9	2.52	12.5	2.59	32.8	1.75	7.4	1.59
8X6 X3/4	33.8	9.94	63.4	2.53	11.7	2.56	30.7	1.76	6.9	1.56
8X6 X11/16	31.2	9.15	58.8	2.54	10.8	2.54	28.6	1.77	6.4	1.54
8X6 X5/8	28.5	8.36	54.1	2.54	9.9	2.52	26.3	1.77	5.9	1.52
8X6 X9/16	25.7	7.56	49.3	2.55	8.9	2.50	24.0	1.78	5.3	1.50
8X6 X1/2	23.0	6.75	44.3	2.56	8.0	2.47	21.7	1.79	4.8	1.47
8X6 X7/16	20.2	5.93	39.2	2.57	7.1	2.45	19.3	1.80	4.2	1.45
8X3½X1	35.7	10.50	66.2	2.51	13.7	3.17	7.8	0.86	3.0	0.92
8X3½X15/16	33.7	9.90	62.9	2.52	12.9	3.14	7.4	0.87	2.9	0.89
8X3½X7/8	31.7	9.30	59.4	2.53	12.2	3.12	7.1	0.87	2.7	0.87
8X3½X13/16	29.6	8.68	55.9	2.54	11.4	3.10	6.7	0.88	2.5	0.85
8X3½X3/4	27.5	8.06	52.3	2.55	10.6	3.07	6.3	0.88	2.3	0.82
8X3½X11/16	25.3	7.43	48.5	2.56	9.8	3.05	5.9	0.89	2.2	0.80
8X3½X5/8	23.2	6.80	44.7	2.57	9.0	3.03	5.4	0.90	2.0	0.78
8X3½X9/16	21.0	6.15	40.8	2.57	8.2	3.00	5.0	0.90	1.8	0.75
8X3½X1/2	18.7	5.50	36.7	2.58	7.3	2.98	4.5	0.91	1.6	0.73
8X3½X7/16	16.5	4.84	32.5	2.59	6.4	2.95	4.1	0.92	1.5	0.70
7X3½X1	32.3	9.50	45.4	2.19	10.6	2.71	7.5	0.89	3.0	0.96
7X3½X15/16	30.5	8.97	43.1	2.19	10.0	2.69	7.2	0.89	2.8	0.94
7X3½X7/8	28.7	8.42	40.8	2.20	9.4	2.66	6.8	0.90	2.6	0.91
7X3½X13/16	26.8	7.87	38.4	2.21	8.8	2.64	6.5	0.91	2.5	0.89
7X3½X3/4	24.9	7.31	36.0	2.22	8.2	2.62	6.1	0.91	2.3	0.87
7X3½X11/16	23.0	6.75	33.5	2.23	7.6	2.60	5.7	0.92	2.1	0.85
7X3½X5/8	21.0	6.17	30.9	2.24	7.0	2.57	5.3	0.93	2.0	0.82
7X3½X9/16	19.1	5.59	28.2	2.25	6.3	2.55	4.9	0.93	1.8	0.80
7X3½X1/2	17.0	5.00	25.4	2.25	5.7	2.53	4.4	0.94	1.6	0.78
7X3½X7/16	15.0	4.40	22.6	2.26	5.0	2.50	4.0	0.95	1.4	0.75
7X3½X3/8	13.0	3.80	19.6	2.27	4.3	2.48	3.5	0.96	1.3	0.73
6X4 X1	30.6	9.00	30.8	1.85	8.0	2.17	10.8	1.09	3.8	1.17
~~[illegible]~~	~~[illegible]~~	~~[illegible]~~	~~[illegible]~~	~~[illegible]~~	~~7.6~~	~~[illegible]~~	~~[illegible]~~	~~[illegible]~~	~~3.6~~	~~[illegible]~~
6X4 X7/8	27.2	7.98	27.7	1.86	7.2	2.12	9.8	1.11	3.4	1.12
6X4 X13/16	25.4	7.47	26.1	1.87	6.7	2.10	9.2	1.11	3.2	1.10
6X4 X3/4	23.6	6.94	24.5	1.88	6.2	2.08	8.7	1.12	3.0	1.08
6X4 X11/16	21.8	6.40	22.8	1.89	5.8	2.06	8.1	1.13	2.8	1.06
6X4 X5/8	20.0	5.86	21.1	1.90	5.3	2.03	7.5	1.13	2.5	1.03
6X4 X9/16	18.1	5.31	19.3	1.90	4.8	2.01	6.9	1.14	2.3	1.01
6X4 X1/2	16.2	4.75	17.4	1.91	4.3	1.99	6.3	1.15	2.1	0.99
6X4 X7/16	14.3	4.18	15.5	1.92	3.8	1.96	5.6	1.16	1.8	0.96
6X4 X3/8	12.3	3.61	13.5	1.93	3.3	1.94	4.9	1.17	1.6	0.94

See "Note" with Table IV, page 354.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XI* (Continued). Properties of Angle-Sections. Unequal Legs

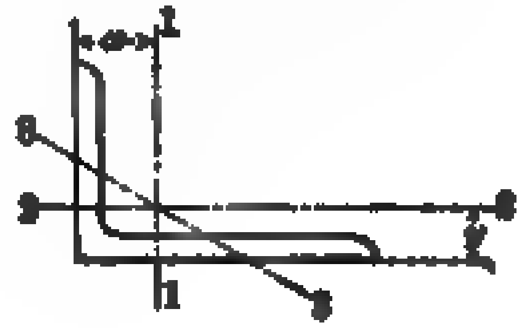


Size	Weight per foot	Area of section	Axis 1-1 *I*	Axis 1-1 *r*	Axis 1-1 *I/c*	Axis 1-1 *x*	Axis 2-2 *I*	Axis 2-2 *r*	Axis 2-2 *I/c*	Axis 2-2 *y*	3-3 r^{min}
in	lb	sq in	in^4	in	in^3	in	in^4	in	in^3	in	in
½X1	28.9	8.50	29.2	1.85	7.8	2.26	7.2	0.92	2.9	1.01	0.74
½X15/16	27.3	8.03	27.8	1.86	7.4	2.24	6.9	0.93	2.7	0.99	0.74
½X⅞	25.7	7.55	26.4	1.87	7.0	2.22	6.6	0.93	2.6	0.97	0.75
½X13/16	24.0	7.06	24.9	1.88	6.6	2.20	6.2	0.94	2.4	0.95	0.75
½X¾	22.4	6.56	23.3	1.89	6.1	2.18	5.8	0.94	2.3	0.93	0.75
½X11/16	20.6	6.06	21.7	1.89	5.6	2.15	5.5	0.95	2.1	0.90	0.75
½X⅝	18.9	5.55	20.1	1.90	5.2	2.13	5.1	0.96	1.9	0.88	0.75
½X9/16	17.1	5.03	18.4	1.91	4.7	2.11	4.7	0.96	1.8	0.86	0.75
½X½	15.3	4.50	16.6	1.92	4.2	2.08	4.3	0.97	1.6	0.83	0.76
½X7/16	13.5	3.97	14.8	1.93	3.7	2.06	3.8	0.98	1.4	0.81	0.76
½X⅜	11.7	3.42	12.9	1.94	3.3	2.04	3.3	0.99	1.2	0.79	0.77
½X5/16	9.8	2.87	10.9	1.95	2.7	2.01	2.9	1.00	1.0	0.76	0.77
4 X⅞	24.2	7.11	16.4	1.52	5.0	1.71	9.2	1.14	3.3	1.21	0.84
4 X13/16	22.7	6.65	15.5	1.53	4.7	1.68	8.7	1.15	3.1	1.18	0.84
4 X¾	21.1	6.19	14.6	1.54	4.4	1.66	8.2	1.15	2.9	1.16	0.84
4 X11/16	19.5	5.72	13.6	1.54	4.1	1.64	7.7	1.16	2.7	1.14	0.84
4 X⅝	17.8	5.23	12.6	1.55	3.7	1.62	7.1	1.17	2.5	1.12	0.84
4 X9/16	16.2	4.75	11.6	1.56	3.4	1.60	6.6	1.18	2.3	1.10	0.85
4 X½	14.5	4.25	10.5	1.57	3.1	1.57	6.0	1.18	2.0	1.07	0.85
4 X7/16	12.8	3.75	9.3	1.58	2.7	1.55	5.3	1.19	1.8	1.05	0.85
4 X⅜	11.0	3.23	8.1	1.59	2.3	1.53	4.7	1.20	1.6	1.03	0.86
½X⅞	22.7	6.67	15.7	1.53	4.9	1.79	6.2	0.97	2.5	1.04	0.75
½X13/16	21.3	6.25	14.8	1.54	4.6	1.77	5.9	0.97	2.4	1.02	0.75
½X¾	19.8	5.81	13.9	1.55	4.3	1.75	5.6	0.98	2.2	1.00	0.75
½X11/16	18.3	5.37	13.0	1.56	4.0	1.72	5.2	0.98	2.1	0.97	0.75
½X⅝	16.8	4.92	12.0	1.56	3.7	1.70	4.8	0.99	1.9	0.95	0.75
½X9/16	15.2	4.47	11.0	1.57	3.3	1.68	4.4	1.00	1.7	0.93	0.75
½X½	13.6	4.00	10.0	1.58	3.0	1.66	4.0	1.01	1.6	0.91	0.75
½X7/16	12.0	3.53	8.9	1.59	2.6	1.63	3.6	1.01	1.4	0.88	0.76
½X⅜	10.4	3.05	7.8	1.60	2.3	1.61	3.2	1.02	1.2	0.86	0.76
½X5/16	8.7	2.56	6.6	1.61	1.9	1.59	2.7	1.03	1.0	0.84	0.76
X13/16	19.9	5.84	14.0	1.55	4.5	1.86	3.7	0.80	1.7	0.86	0.64
X¾	18.5	5.44	13.2	1.55	4.2	1.84	3.5	0.80	1.6	0.84	0.64
X11/16	17.1	5.03	12.3	1.56	3.9	1.82	3.3	0.81	1.5	0.82	0.64
X⅝	15.7	4.61	11.4	1.57	3.5	1.80	3.1	0.81	1.4	0.80	0.64
X9/16	14.3	4.18	10.4	1.58	3.2	1.77	2.8	0.82	1.3	0.77	0.65
X½	12.8	3.75	9.5	1.59	2.9	1.75	2.6	0.83	1.1	0.75	0.65
X7/16	11.3	3.31	8.4	1.60	2.6	1.73	2.3	0.84	1.0	0.73	0.65
X⅜	9.8	2.86	7.4	1.61	2.2	1.70	2.0	0.84	0.89	0.70	0.65
X5/16	8.2	2.40	6.3	1.61	1.9	1.68	1.8	0.85	0.75	0.68	0.66

See "Note" with Table IV, page 354.

*From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XI* (Continued). Properties of Angle-Sections. Unequal Legs

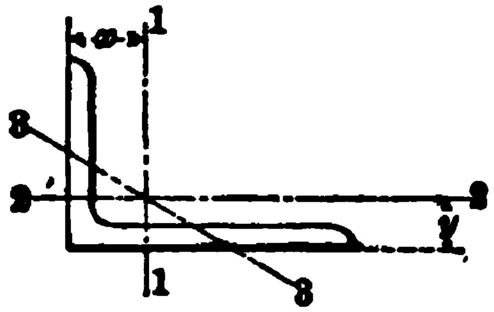


4½	18.5	5.43	10.3	1.38	3.6	1.65	3.6	0.81	1.7	0.90	0
4½	17.3	5.06	9.7	1.39	3.4	1.63	3.4	0.82	1.6	0.88	0
4½	16.0	4.68	9.1	1.39	3.1	1.60	3.2	0.83	1.5	0.85	0
4½	14.7	4.30	8.4	1.40	2.9	1.58	3.0	0.83	1.4	0.83	0
4½	13.3	3.90	7.8	1.41	2.6	1.56	2.8	0.85	1.3	0.81	0
4½	11.9	3.50	7.0	1.42	2.4	1.54	2.5	0.85	1.1	0.79	0
4½	10.6	3.09	6.3	1.43	2.1	1.51	2.3	0.85	1.0	0.76	0
4½	9.1	2.67	5.5	1.44	1.8	1.49	2.0	0.86	0.88	0.74	0
4½	7.7	2.25	4.7	1.44	1.5	1.47	1.7	0.87	0.75	0.72	0
4	18.5	5.43	7.8	1.19	2.9	1.36	5.5	1.01	2.3	1.11	0
4	17.3	5.06	7.3	1.20	2.8	1.34	5.2	1.01	2.1	1.09	0
4	16.0	4.68	6.9	1.21	2.6	1.32	4.9	1.02	2.0	1.07	0
4	14.7	4.30	6.4	1.22	2.4	1.29	4.5	1.03	1.8	1.04	0
4	13.3	3.90	5.9	1.23	2.1	1.27	4.2	1.03	1.7	1.02	0
4	11.9	3.50	5.3	1.23	1.9	1.25	3.8	1.04	1.5	1.00	0
4	10.6	3.09	4.8	1.24	1.7	1.23	3.4	1.05	1.3	0.98	0
4	9.1	2.67	4.2	1.25	1.5	1.21	3.0	1.06	1.2	0.96	0
4	7.7	2.25	3.6	1.26	1.3	1.18	2.6	1.07	1.0	0.93	0
4 ×3 ×13/16	17.1	5.03	7.3	1.21	2.9	1.44	3.5	0.83	1.7	0.94	0
4 ×3 ×3/4	16.0	4.69	6.9	1.22	2.7	1.42	3.3	0.84	1.6	0.92	0
4 ×3 ×11/16	14.8	4.34	6.5	1.22	2.5	1.39	3.1	0.84	1.5	0.89	0
4 ×3 ×5/8	13.6	3.98	6.0	1.23	2.3	1.37	2.9	0.85	1.4	0.87	0
4 ×3 ×9/16	12.4	3.62	5.6	1.24	2.1	1.35	2.7	0.86	1.2	0.85	0
4 ×3 ×1/2	11.1	3.25	5.0	1.25	1.9	1.33	2.4	0.86	1.1	0.83	0
4 ×3 ×7/16	9.8	2.87	4.5	1.25	1.7	1.30	2.2	0.87	1.0	0.80	0
4 ×3 ×3/8	8.5	2.48	4.0	1.26	1.5	1.28	1.9	0.88	0.87	0.78	0
4 ×3 ×5/16	7.2	2.09	3.4	1.27	1.2	1.26	1.7	0.89	0.74	0.76	0
4 ×3 ×1/4	5.8	1.69	2.8	1.28	1.0	1.24	1.4	0.89	0.60	0.74	0
3½×3 ×13/16	15.8	4.62	5.0	1.04	2.2	1.23	3.3	0.85	1.7	0.98	0
3½×3 ×3/4	14.7	4.31	4.7	1.04	2.1	1.21	3.1	0.85	1.5	0.96	0
3½×3 ×11/16	13.6	4.00	4.4	1.05	1.9	1.19	3.0	0.86	1.4	0.94	0
3½×3 ×5/8	12.5	3.67	4.1	1.06	1.8	1.17	2.8	0.87	1.3	0.92	0
3½×3 ×9/16	11.4	3.34	3.8	1.07	1.6	1.15	2.5	0.87	1.2	0.90	0
3½×3 ×1/2	10.2	3.00	3.5	1.07	1.5	1.13	2.3	0.88	1.1	0.88	0
3½×3 ×7/16	9.1	2.65	3.1	1.08	1.3	1.10	2.1	0.89	0.98	0.85	0
3½×3 ×3/8	7.9	2.30	2.7	1.09	1.1	1.08	1.8	0.90	0.85	0.83	0
3½×3 ×5/16	6.6	1.93	2.3	1.10	0.96	1.06	1.6	0.90	0.72	0.81	0
3½×3 ×1/4	5.4	1.56	1.9	1.11	0.78	1.04	1.3	0.91	0.58	0.79	0
3½×2½×11/16	12.5	3.65	4.1	1.06	1.9	1.27	1.7	0.69	0.99	0.77	0
3½×2½×5/8	11.5	3.36	3.8	1.07	1.7	1.25	1.6	0.69	0.92	0.75	0
3½×2½×9/16	10.4	3.06	3.6	1.08	1.6	1.23	1.5	0.70	0.84	0.73	0
3½×2½×1/2	9.4	2.75	3.2	1.09	1.4	1.20	1.4	0.70	0.76	0.70	0
3½×2½×7/16	8.3	2.43	2.9	1.09	1.3	1.18	1.2	0.71	0.68	0.68	0
3½×2½×3/8	7.2	2.11	2.6	1.10	1.1	1.16	1.1	0.72	0.59	0.66	0
3½×2½×5/16	6.1	1.78	2.2	1.11	0.93	1.14	0.94	0.73	0.50	0.64	0
3½×2½×1/4	4.9	1.44	1.8	1.12	0.75	1.11	0.78	0.74	0.41	0.61	0

See "Note" with Table IV, page 354.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XI* (Continued). Properties of Angle-Sections. Unequal Legs

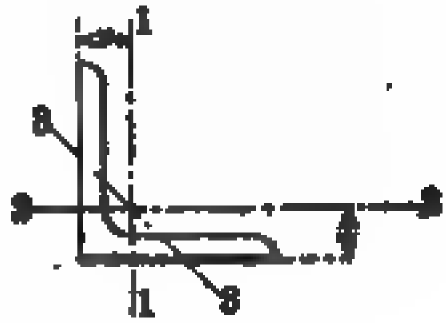


Size	Weight per foot	Area of section	Axis 1-1				Axis 2-2				Axis 3-3
			I	r	I/c	x	I	r	I/c	y	r_{min}
in	lb	sq in	in⁴	in	in³	in	in⁴	in	in³	in	in
× 2½ × 9/16	9.5	2.78	2.3	0.91	1.2	1.02	1.4	0.72	0.82	0.77	0.52
× 2½ × ½	8.5	2.50	2.1	0.91	1.0	1.00	1.3	0.72	0.74	0.75	0.52
× 2½ × 7/16	7.6	2.21	1.9	0.92	0.93	0.98	1.2	0.73	0.66	0.73	0.52
× 2½ × ⅜	6.6	1.92	1.7	0.93	0.81	0.96	1.0	0.74	0.58	0.71	0.52
× 2½ × 5/16	5.6	1.62	1.4	0.94	0.69	0.93	0.90	0.74	0.49	0.68	0.53
× 2½ × ¼	4.5	1.31	1.2	0.95	0.56	0.91	0.74	0.75	0.40	0.66	0.53
× 2 × ½	7.7	2.25	1.9	0.92	1.0	1.08	0.67	0.55	0.47	0.58	0.43
× 2 × 7/16	6.8	2.00	1.7	0.93	0.89	1.06	0.61	0.55	0.42	0.56	0.43
× 2 × ⅜	5.9	1.73	1.5	0.94	0.78	1.04	0.54	0.56	0.37	0.54	0.43
× 2 × 5/16	5.0	1.47	1.3	0.95	0.66	1.02	0.47	0.57	0.32	0.52	0.43
× 2 × ¼	4.1	1.19	1.1	0.95	0.54	0.99	0.39	0.57	0.25	0.49	0.43
× 2 × ½	6.8	2.00	1.1	0.75	0.70	0.88	0.64	0.56	0.46	0.63	0.42
× 2 × 7/16	6.1	1.78	1.0	0.76	0.62	0.85	0.58	0.57	0.41	0.60	0.42
× 2 × ⅜	5.3	1.55	0.91	0.77	0.55	0.83	0.51	0.58	0.36	0.58	0.42
× 2 × 5/16	4.5	1.31	0.79	0.78	0.47	0.81	0.45	0.58	0.31	0.56	0.42
× 2 × ¼	3.62	1.06	0.65	0.78	0.38	0.79	0.37	0.59	0.25	0.54	0.42
× 2 × 3/16	2.75	0.81	0.51	0.79	0.29	0.76	0.29	0.60	0.20	0.51	0.43
× 2 × ⅛	1.86	0.55	0.35	0.80	0.20	0.74	0.20	0.61	0.13	0.49	0.43
× 1½ × 5/16	3.92	1.15	0.71	0.79	0.44	0.90	0.19	0.41	0.17	0.40	0.32
× 1½ × ¼	3.19	0.94	0.59	0.79	0.36	0.88	0.16	0.41	0.14	0.38	0.32
× 1½ × 3/16	2.44	0.72	0.46	0.80	0.28	0.85	0.13	0.42	0.11	0.35	0.33
× 1½ × ½	5.6	1.63	0.75	0.68	0.54	0.86	0.26	0.40	0.26	0.48	0.32
× 1½ × 7/16	5.0	1.45	0.68	0.69	0.48	0.83	0.24	0.41	0.23	0.46	0.32
× 1½ × ⅜	4.4	1.27	0.61	0.69	0.42	0.81	0.21	0.41	0.20	0.44	0.32
× 1½ × 5/16	3.66	1.07	0.53	0.70	0.36	0.79	0.19	0.42	0.17	0.42	0.32
× 1½ × ¼	2.98	0.88	0.44	0.71	0.30	0.77	0.16	0.42	0.14	0.39	0.32
× 1½ × 3/16	2.28	0.67	0.34	0.72	0.23	0.75	0.12	0.43	0.11	0.37	0.33
× 1½ × ⅜	3.99	1.17	0.43	0.61	0.34	0.71	0.21	0.42	0.20	0.46	0.32
× 1½ × 5/16	3.39	1.00	0.38	0.62	0.29	0.69	0.18	0.42	0.17	0.44	0.32
× 1½ × ¼	2.77	0.81	0.32	0.62	0.24	0.66	0.15	0.43	0.1[illegible]	0.41	0.32
× 1½ × 3/16	2.12	0.62	0.25	0.63	0.18	0.64	0.12	0.44	0.11	0.39	0.32
× 1½ × ⅛	1.44	0.42	0.17	0.64	0.13	0.62	0.09	0.45	0.08	0.37	0.33
× 1¼ × ¼	2.55	0.75	0.30	0.63	0.23	0.71	0.09	0.34	0.10	0.33	0.27
× 1¼ × 3/16	1.96	0.57	0.23	0.64	0.18	0.69	0.07	0.35	0.08	0.31	0.27
× 1¼ × ¼	2.34	0.69	0.20	0.54	0.18	0.60	0.09	0.35	0.10	0.35	0.27
× 1¼ × 3/16	1.80	0.53	0.16	0.55	0.14	0.58	0.07	0.36	0.08	0.33	0.27
× 1¼ × ⅛	1.23	0.36	0.11	0.56	0.09	0.56	0.05	0.37	0.05	0.31	0.27
× 1¼ × 5/16	2.59	0.76	0.16	0.45	0.16	0.52	0.10	0.35	0.11	0.40	0.26
× 1¼ × ¼	2.13	0.63	0.13	0.46	0.13	0.50	0.08	0.36	0.09	0.38	0.26
× 1¼ × 3/16	1.64	0.48	0.10	0.46	0.10	0.48	0.07	0.37	0.07	0.35	0.26

…e " Note " with Table IV, page 354.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XII.* Properties of Angle-Sections. Equal Legs

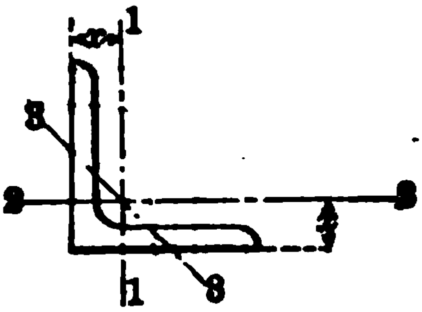


Size	Weight per foot	Area of section	Axis 1-1 and Axis 2-2				Axis 3-3
			I	r	I/c	x	r_{min}
in	lb	sq in	in⁴	in	in³	in	in
8×8×1⅛	56.9	[illegible] 73	98.0	2.42	17.5	2.41	1.5
8×8×1 1/16	54.0	15.87	93.5	2.43	16.7	2.39	1.5
8×8×1	51.0	15.00	89.0	2.44	15.8	2.37	1.5
8×8×15/16	48.1	14.12	84.3	2.44	14.9	2.34	1.5
8×8×⅞	45.0	13.23	79.6	2.45	14.0	2.32	1.5
8×8×13/16	42.0	12.34	74.7	2.46	13.1	2.30	1.5
8×8×¾	38.9	11.44	69.7	2.47	12.2	2.28	1.5
8×8×11/16	35.8	10.53	64.6	2.48	11.2	2.25	1.5
8×8×⅝	32.7	9.61	59.4	2.49	10.3	2.23	1.5
8×8×9/16	[illegible]	8.68	54.1	2.50	9.3	2.21	1.5
8×8×½	[illegible].4	7.75	48.6	2.51	8.4	2.19	1.5
6×6×1	37.4	11.00	35.5	1.80	8.6	1.86	1.1
6×6×15/16	35.3	10.37	33.7	1.80	8.1	1.84	1.1
6×6×⅞	33.1	9.73	31.9	1.81	7.6	1.82	1.1
6×6×13/16	31.0	9.09	30.1	1.82	7.2	1.80	1.1
6×6×¾	28.7	8.44	28.2	1.83	6.7	1.78	1.1
6×6×11/16	26.5	7.78	26.2	1.83	6.2	1.75	1.1
6×6×⅝	24.2	7.11	24.2	1.84	5.7	1.73	1.1
6×6×9/16	21.9	6.43	22.1	1.85	5.1	1.71	1.1
6×6×½	19.6	5.75	19.9	1.86	4.6	1.68	1.1
6×6×7/16	17.2	5.06	17.7	1.87	4.1	1.66	1.1
6×6×⅜	14.9	4.36	15.4	1.88	3.5	1.64	1.1
	30.6	9.00	19.6	1.48	5.8	1.61	0.9
	28.9	8.50	18.7	1.48	5.5	1.59	0.9
	27.2	7.98	17.8	1.49	5.2	1.57	0.9
	25.4	7.47	16.8	1.50	4.9	1.55	0.9
	23.6	6.94	15.7	1.50	4.5	1.52	0.9
	21.8	6.40	14.7	1.51	4.2	1.50	0.9
	20.0	5.86	13.6	1.52	3.9	1.48	0.9
	18.1	5.31	12.4	1.53	3.5	1.46	0.9
	16.2	4.75	11.3	1.54	3.2	1.43	0.9
	14.3	4.18	10.0	1.55	2.8	1.41	0.9
	12.3	3.61	8.7	1.56	2.4	1.39	0.9
4×4×13/16	19.9	5.84	8.1	1.18	3.0	1.29	0.
4×4×¾	18.5	5.44	7.7	1.19	2.8	1.27	0.
4×4×11/16	17.1	5.03	7.2	1.19	2.6	1.25	0.
4×4×⅝	15.7	4.61	6.7	1.20	2.4	1.23	0.
4×4×9/16	14.3	4.18	6.1	1.21	2.2	1.21	0.
4×4×½	12.8	3.75	5.6	1.22	2.0	1.18	0.
4×4×7/16	11.3	3.31	5.0	1.23	1.8	1.16	0.
4×4×⅜	9.8	2.86	4.4	1.23	1.5	1.14	0.
4×4×5/16	8.2	2.40	3.7	1.24	1.3	1.12	0.
4×4×¼	6.6	1.94	3.0	1.25	1.0	1.09	0.

See "Note" with Table IV, page 354.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XII* (Continued). Properties of Angle-Sections. Equal Legs

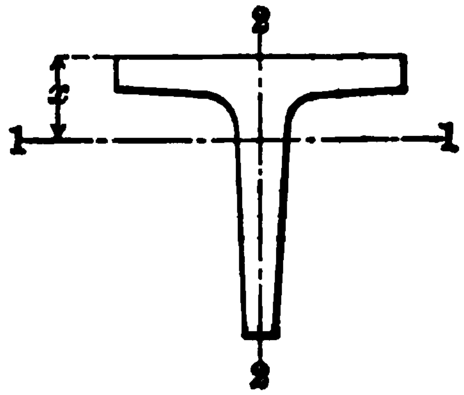


Size	Weight per foot	Area of section	Axis 1-1 and Axis 2-2 I	Axis 1-1 and Axis 2-2 r	Axis 1-1 and Axis 2-2 I/c	Axis 1-1 and Axis 2-2 x	Axis 3-3 r_{min}
in	lb	sq in	in⁴	in	in³	in	in
3½×13/16	17.1	5.03	5.3	1.02	2.3	1.17	0.67
3½×3/4	16.0	4.69	5.0	1.03	2.1	1.15	0.67
3½×11/16	14.8	4.34	4.7	1.04	2.0	1.12	0.67
3½×5/8	13.6	3.98	4.3	1.04	1.8	1.10	0.68
3½×9/16	12.4	3.62	4.0	1.05	1.6	1.08	0.68
3½×1/2	11.1	3.25	3.6	1.06	1.5	1.06	0.68
3½×7/16	9.8	2.87	3.3	1.07	1.3	1.04	0.68
3½×3/8	8.5	2.48	2.9	1.07	1.2	1.01	0.69
3½×5/16	7.2	2.09	2.5	1.08	0.98	0.99	0.69
3½×1/4	5.8	1.69	2.0	1.09	0.79	0.97	0.69
3 ×5/8	11.5	3.36	2.6	0.88	1.3	0.98	0.57
3 ×9/16	10.4	3.06	2.4	0.89	1.2	0.95	0.58
3 ×1/2	9.4	2.75	2.2	0.90	1.1	0.93	0.58
3 ×7/16	8.3	2.43	2.0	0.91	0.95	0.91	0.58
3 ×3/8	7.2	2.11	1.8	0.91	0.83	0.89	0.58
3 ×5/16	6.1	1.78	1.5	0.92	0.71	0.87	0.59
3 ×1/4	4.9	1.44	1.2	0.93	0.58	0.84	0.59
2½×1/2	7.7	2.25	1.2	0.74	0.73	0.81	0.47
2½×7/16	6.8	2.00	1.1	0.75	0.65	0.78	0.48
2½×3/8	5.9	1.73	0.98	0.75	0.57	0.76	0.48
2½×5/16	5.0	1.47	0.85	0.76	0.48	0.74	0.49
2½×1/4	4.1	1.19	0.70	0.77	0.39	0.72	0.49
2½×3/16	3.07	0.90	0.55	0.78	0.30	0.69	0.49
2½×1/8	2.08	0.61	0.38	0.79	0.20	0.67	0.50
2 ×7/16	5.3	1.56	0.54	0.59	0.40	0.66	0.39
2 ×3/8	4.7	1.36	0.48	0.59	0.35	0.64	0.39
2 ×5/16	3.92	1.15	0.42	0.60	0.30	0.61	0.39
2 ×1/4	3.19	0.94	0.35	0.61	0.25	0.59	0.39
2 ×3/16	2.44	0.71	0.28	0.62	0.19	0.57	0.40
2 ×1/8	1.65	0.48	0.19	0.63	0.13	0.55	0.40
1¾×7/16	4.6	1.34	0.35	0.51	0.30	0.59	0.33
1¾×3/8	3.99	1.17	0.31	0.51	0.26	0.57	0.34
1¾×5/16	3.39	1.00	0.27	0.52	0.23	0.55	0.34
1¾×1/4	2.77	0.81	0.23	0.53	0.19	0.53	0.34
1¾×3/16	2.12	0.62	0.18	0.54	0.14	0.51	0.35
1¾×1/8	1.44	0.42	0.13	0.55	0.10	0.48	0.35
1½×3/8	3.35	0.98	0.19	0.44	0.19	0.51	0.29
1½×5/16	2.86	0.84	0.16	0.44	0.16	0.49	0.29
1½×1/4	2.34	0.69	0.14	0.45	0.13	0.47	0.29
1½×3/16	1.80	0.53	0.11	0.46	0.10	0.44	0.29
1½×1/8	1.23	0.36	0.08	0.46	0.07	0.42	0.30
1¼×5/16	2.33	0.68	0.09	0.36	0.11	0.42	0.24
1¼×1/4	1.92	0.56	0.08	0.37	0.09	0.40	0.24
1¼×3/16	1.48	0.43	0.06	0.38	0.07	0.38	0.24
1¼×1/8	1.01	0.30	0.04	0.38	0.05	0.35	0.25
1 ×1/4	1.49	0.44	0.04	0.29	0.06	0.34	0.19
1 ×3/16	1.16	0.34	0.03	0.30	0.04	0.32	0.19
1 ×1/8	0.80	0.23	0.02	0.31	0.03	0.30	0.19

Note * with Table IV, page 354.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XIII.* Properties of T Sections. Flange and Stem Equal

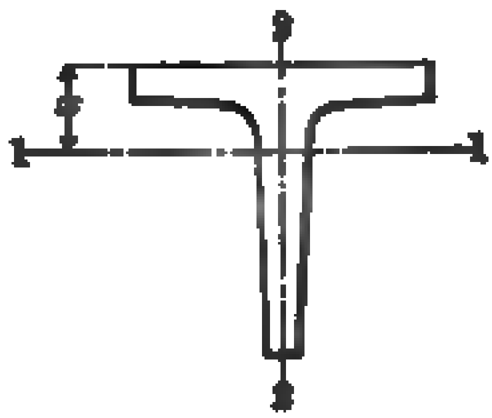


Size				Weight per foot	Area of section	Axis 1-1				Axis 2	
Flange	Stem	Minimum thickness				I	r	I/c	x	I	r
		Flange	Stem								
in	in	in	in	lb	sq in	in⁴	in	in³	in	in⁴	in
6½	6½	0.40	0.45	19.8	5.80	23.5	2.01	5.0	1.76	10.1	1.3
4	4	½	½	13.5	3.97	5.7	1.20	2.0	1.18	2.8	0.8
4	4	⅜	⅜	10.5	3.09	4.5	1.21	1.6	1.13	2.1	0.8
3½	3½	½	½	11.7	3.44	3.7	1.04	1.5	1.05	1.9	0.7
3½	3½	⅜	⅜	9.2	2.68	3.0	1.05	1.2	1.01	1.4	0.7
3	3	½	½	9.9	2.91	2.3	0.88	1.1	0.93	1.2	0.6
3	3	7/16	7/16	8.9	2.59	2.1	0.89	0.98	0.91	1.0	0.6
3	3	⅜	⅜	7.8	2.27	1.8	0.90	0.86	0.88	0.90	0.6
3	3	5/16	5/16	6.7	1.95	1.6	0.90	0.74	0.86	0.75	0.6
2½	2½	⅜	⅜	6.4	1.87	1.0	0.74	0.59	0.76	0.52	0.5
2½	2½	5/16	5/16	5.5	1.60	0.88	0.74	0.50	0.74	0.44	0.5
2¼	2¼	5/16	5/16	4.9	1.43	0.65	0.67	0.41	0.68	0.33	0.4
2¼	2¼	¼	¼	4.1	1.19	0.52	0.66	0.32	0.65	0.25	0.4
2	2	5/16	5/16	4.3	1.26	0.44	0.59	0.31	0.61	0.23	0.4
2	2	¼	¼	3.56	1.05	0.37	0.59	0.26	0.59	0.18	0.4
1¾	1¾	¼	¼	3.09	0.91	0.23	0.51	0.19	0.54	0.12	0.3
1½	1½	¼	¼	2.47	0.73	0.15	0.45	0.14	0.47	0.08	0.3
1½	1½	3/16	3/16	1.94	0.57	0.11	0.45	0.11	0.44	0.06	0.3
1¼	1¼	¼	¼	2.02	0.59	0.08	0.37	0.10	0.40	0.05	0.2
1¼	1¼	3/16	3/16	1.59	0.47	0.06	0.37	0.07	0.38	0.03	0.1
1	1	3/16	3/16	1.25	0.37	0.03	0.29	0.05	0.32	0.02	0.1
1	1	⅛	⅛	0.89	0.26	0.02	0.30	0.03	0.29	0.01	0.1

See "Note" with Table IV, page 354.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XIV.* Properties of T Sections. Flange and Stem Unequal



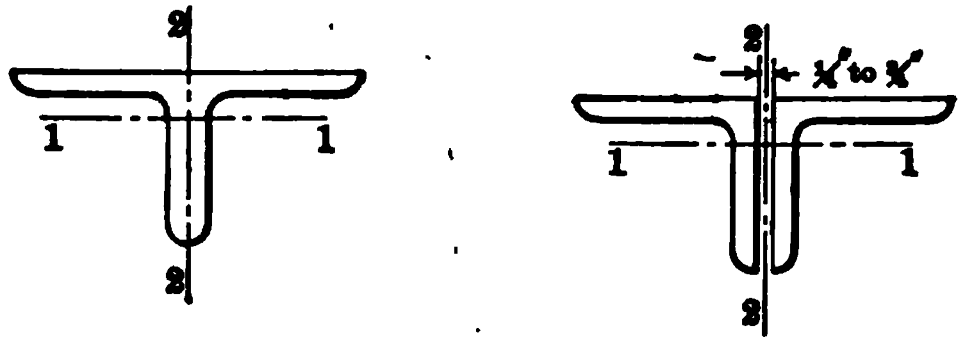
Stem	Thickness: Flange	Thickness: Stem	Weight per foot	Area of section	Axis 1-1: I	Axis 1-1: r	Axis 1-1: I/c	Axis 1-1: x	Axis 2-2: I	Axis 2-2: r	Axis 2-2: I/c
in	in	in	lb	sq in	in⁴	in	in³	in	in⁴	in	in³
3	½	13/32	11.5	3.37	2.4	0.84	1.1	0.76	3.9	1.10	1.6
2½	⅜	7/16	10.9	3.18	1.5	0.68	0.76	0.63	4.1	1.14	1.6
3½	7/16	11/16	15.7	4.60	5.1	1.05	2.1	1.11	3.7	0.90	1.7
3	⅜	[illegible]	9.8	2.88	2.1	0.84	0.91	0.74	3.0	1.02	1.3
3	5/16	[illegible]	8.4	2.46	1.8	0.85	0.76	0.71	2.5	1.01	1.1
2½	⅜	[illegible]	9.2	2.68	1.2	0.67	0.63	0.59	3.0	1.05	1.3
2½	5/16	[illegible]	7.8	2.29	1.0	0.68	0.54	0.57	2.5	1.05	1.1
5	½	[illegible]	15.3	4.50	10.8	1.55	3.1	1.56	2.8	0.79	1.4
5	⅜	[illegible]	11.9	3.49	8.5	1.56	2.4	1.51	2.1	0.78	1.1
4½	½	[illegible]	14.4	4.23	7.9	1.37	2.5	1.37	2.8	0.81	1.4
4½	⅜	[illegible]	11.2	3.29	6.3	1.39	2.0	1.31	2.1	0.80	1.1
3	⅜	[illegible]	9.2	2.68	2.0	0.86	0.90	0.78	2.1	0.89	1.1
3	5/16	[illegible]	7.8	2.29	1.7	0.87	0.77	0.75	1.8	0.88	0.88
2½	⅜	[illegible]	8.5	2.48	1.2	0.69	0.62	0.62	2.1	0.92	1.0
2½	5/16	[illegible]	7.2	2.12	1.0	0.69	0.53	0.60	1.8	0.91	0.88
2	⅜	[illegible]	7.8	2.27	0.60	0.52	0.40	0.48	2.1	0.96	1.1
2	5/16	[illegible]	6.7	1.95	0.53	0.52	0.34	0.46	1.8	0.95	0.88
4	½	[illegible]	12.6	3.70	5.5	1.21	2.0	1.24	1.9	0.72	1.1
4	⅜	[illegible]	9.8	2.88	4.3	1.23	1.5	1.19	1.4	0.70	0.81
3	½	[illegible]	10.8	3.17	2.4	0.87	1.1	0.88	1.9	0.77	1.1
3	⅜	[illegible]	8.5	2.48	1.9	0.88	0.89	0.83	1.4	0.75	0.81
3	5/16	[illegible]	7.5	2.20	1.8	0.91	0.85	0.85	1.2	0.74	0.68
4	½	[illegible]	11.7	3.44	5.2	1.23	1.9	1.32	1.2	0.59	0.81
4	7/16	[illegible]	10.5	3.06	4.7	1.23	1.7	1.29	1.1	0.59	0.70
4	⅜	[illegible]	9.2	2.68	4.1	1.24	1.5	1.27	0.90	0.58	0.60
3½	½	[illegible]	10.8	3.17	3.5	1.06	1.5	1.12	1.2	0.62	0.80
3½	[illegible]	[illegible]	9.7	2.83	3.2	1.06	1.3	1.10	1.0	0.60	0.69
3½		[illegible]	8.5	2.48	2.8	1.07	1.2	1.07	0.93	0.61	0.[illegible]
2½		[illegible]	7.1	2.07	1.1	0.72	0.60	0.71	0.89	0.66	0.59
2½		[illegible]	6.1	1.77	0.94	0.73	0.52	0.68	0.75	0.65	0.50
3		[illegible]	7.1	2.07	1.7	0.91	0.84	0.95	0.53	0.51	0.42
3		[illegible]	6.1	1.77	1.5	0.92	0.72	0.92	0.44	0.50	0.35
1¾		[illegible]	2.87	0.84	0.08	0.31	0.09	0.32	0.29	0.58	0.23
1½		[illegible]	3.09	0.91	0.16	0.42	0.15	0.42	0.18	0.45	0.18
2		[illegible]	2.45	0.72	0.27	0.61	0.19	0.63	0.06	0.92	0.08
1¼		[illegible]	1.25	0.37	0.05	0.37	0.05	0.33	0.04	0.32	0.05
⅞	[illegible]	[illegible]	0.88	0.26	0.01	0.16	0.01	0.16	0.02	0.31	0.04

"Note" with Table IV, page 354.

*From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XV.* Properties of Double-Angle Sections. Equal Legs

ANGLES PLACED BACK TO BACK



Single angle: Size, in	Single angle: Weight per foot, lb	Two angles: Area, sq in	Radii of gyration, r, in inches: Axis 1-1	Axis 2-2: In contact	Axis 2-2: ¼-in apart	Axis 2-2: ⅜-in apart	Axis 2-2: ½-in apart
8 × 8 × 1⅛	56.9	33.46	2.42	3.42	3.51	3.55	3.60
1 3/16	42.0	24.68	2.46	3.37	3.46	3.50	3.55
½	26.4	15.50	2.50	3.33	3.41	3.45	3.50
6 × 6 × 1	37.4	22.00	1.80	2.59	2.68	2.72	2.77
11/16	26.5	15.56	1.83	2.54	2.63	2.67	2.71
⅜	14.9	8.72	1.88	2.49	2.58	2.62	2.66
5 × 5 × 1	30.6	18.00	1.48	2.19	2.28	2.33	2.38
11/16	21.8	12.80	1.51	2.13	2.22	2.26	2.31
⅜	12.3	7.22	1.56	2.09	2.17	2.21	2.26
4 × 4 × 13/16	19.9	11.68	1.18	1.75	1.85	1.89	1.94
¼	6.6	3.88	1.25	1.66	1.75	1.79	1.84
3½ × 3½ × 13/16	17.1	10.06	1.02	1.55	1.65	1.70	1.75
¼	5.8	3.38	1.09	1.46	1.55	1.59	1.64
3 × 3 × ⅝	11.5	6.72	0.88	1.32	1.41	1.46	1.51
¼	4.9	2.88	0.93	1.25	1.34	1.38	1.43
2½ × 2½ × ½	7.7	4.50	0.74	1.09	1.19	1.24	1.29
¼	4.1	2.38	0.77	1.05	1.14	1.19	1.24
2 × 2 × 7/16	5.3	3.12	0.59	0.88	0.98	1.03	1.08
¼	3.19	1.88	0.61	0.85	0.94	0.99	1.04

This table and the two following are employed in computing the safe resista compressive stress of two angles, back to back, used as struts or as the compre chords of roof-trusses, etc., by the following rule:

Obtain from the compression-formula in use the allowed stress per squar corresponding to the ratio of slenderness of the section, and multiply that val the area. The result will be the allowable compressive stress.

Example 1. Section given. Required the safe load in compression, as per fo $S = 19\,000 - 100\,l/r$, on a strut composed of two angles, 4 by 4 by ¼ in, back to with an unsupported length of 9 ft.

Area of section, $A = 3.88$ sq in; least radius of gyration, $r = 1.25$ in.

Ratio of slenderness, $l/r = 9 \times 12 \div 1.25 = 86.4$.

Allowed unit stress, $S = 19\,000 - 100 \times 86.4 = 10\,360$ lb per sq in.

Safe load, $AS = 3.88 \times 10\,360 = 40\,200$ lb.

Example 2. Stress given. Required a section for a member in compre 12 ft 3 in long, made of two angles separated by ½-in gusset-plates, to resist a stress of 35 000 lb; ratio of slenderness not to exceed 120.

Assume two angles, 5 by 3 by 5/16 in, long legs back to back.

Area of section, $A = 4.80$ sq in; least radius of gyration, $r = 1.26$ in.

Ratio of slenderness, $l/r = 12.25 \times 12 \div 1.26 = 116.7$.

Allowed unit stress, $S = 19\,000 - 100 \times 116.7 = 7\,330$ lb per sq in.

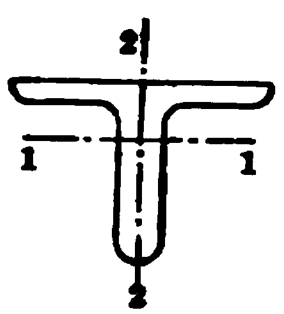
Safe load, $AS = 4.80 \times 7\,330 = 35\,200$ lb.

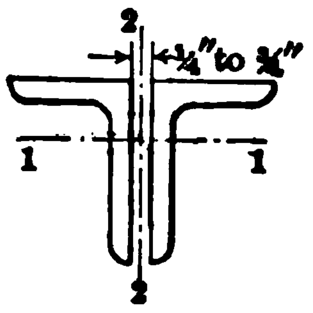
In the first case the least radius of gyration is that about the axis 1-1; in the s case, about the axis 2-2; in all cases the least radius of gyration determines th of slenderness and therewith the allowed safe compressive stress. In all cases the two angles are to be secured together by stay-rivets, so spaced as to insur the section acts as a unit. The ratio of slenderness of any single angle between must always be less than that of the strut or compression-chord.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XVI.* Properties of Double-Angle Sections. Long Legs Vertical

ANGLES PLACED BACK TO BACK



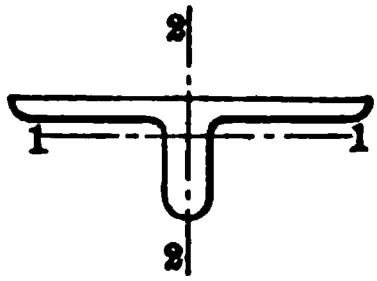


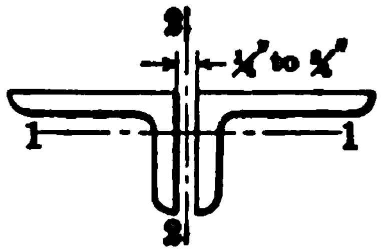
Single angle		Two angles	Radii of gyration, r, in inches					
				Axis 2–2				
Size, in	Weight per foot, lb	Area, sq in	Axis 1–1	In contact	¼-in apart	⅜-in apart	½-in apart	¾-in apart
×6 ×1	44.2	26.00	2.49	2.39	2.48	2.52	2.57	2.66
¾	33.8	19.88	2.53	2.35	2.44	2.48	2.52	2.61
7/16	20.2	11.86	2.57	2.31	2.39	2.43	2.48	2.56
×3½×1	35.7	21.00	2.51	1.26	1.35	1.40	1.45	1.55
¾	27.5	16.12	2.55	1.20	1.29	1.34	1.39	1.49
7/16	16.5	9.68	2.59	1.15	1.23	1.28	1.32	1.41
×3½×1	32.3	19.00	2.19	1.31	1.40	1.45	1.50	1.60
11/16	23.0	13.50	2.23	1.25	1.34	1.39	1.44	1.53
⅜	13.0	7.60	2.27	1.20	1.28	1.33	1.37	1.46
×4 ×1	30.6	18.00	1.85	1.60	1.69	1.74	1.79	1.89
11/16	21.8	12.80	1.89	1.55	1.63	1.68	1.73	1.82
⅜	12.3	7.22	1.93	1.50	1.58	1.62	1.67	1.76
×3½×1	28.9	17.00	1.85	1.37	1.47	1.51	1.56	1.66
11/16	20.6	12.12	1.89	1.32	1.41	1.45	1.49	1.60
5/16	9.8	5.74	1.95	1.25	1.33	1.37	1.42	1.50
×4 ×¾	24.2	14.22	1.58	1.66	1.76	1.80	1.85	1.95
⅜	11.0	6.46	1.59	1.58	1.66	1.70	1.75	1.85
×3½×¾	22.7	13.34	1.53	1.42	1.51	1.56	1.61	1.71
5/16	8.7	5.12	1.61	1.33	1.41	1.45	1.50	1.59
×3 ×13/16	19.9	11.68	1.55	1.18	1.27	1.32	1.37	1.47
5/16	8.2	4.80	1.61	1.09	1.17	1.22	1.26	1.35
×3 ×13/16	18.5	10.86	1.38	1.21	1.31	1.36	1.41	1.51
5/16	7.7	4.50	1.44	1.13	1.22	1.26	1.30	1.40
×3½×13/16	18.5	10.86	1.19	1.50	1.59	1.64	1.69	1.79
5/16	7.7	4.50	1.26	1.42	1.51	1.55	1.60	1.69
×3 ×13/16	17.1	10.06	1.21	1.25	1.35	1.40	1.45	1.55
¼	5.8	3.38	1.28	1.16	1.24	1.28	1.33	1.43
×3 ×13/16	15.8	9.24	1.04	1.30	1.40	1.45	1.50	1.60
¼	5.4	3.12	1.11	1.20	1.29	1.34	1.38	1.48
×2½×11/16	12.5	7.30	1.06	1.03	1.13	1.18	1.23	1.33
¼	4.9	2.88	1.12	0.95	1.04	1.09	1.13	1.23
×2½×9/16	9.5	5.56	0.91	1.05	1.15	1.20	1.25	1.35
¼	4.5	2.64	0.95	1.00	1.09	1.13	1.18	1.28
×2 ×½	7.7	4.50	0.92	0.80	0.89	0.94	1.00	1.10
¼	4.1	2.38	0.95	0.74	0.84	0.88	0.93	1.03
×2 ×½	6.8	4.00	0.75	0.84	0.94	0.99	1.04	1.15
¼	3.62	2.12	0.78	0.80	0.89	0.93	0.98	1.08

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XVII.* Properties of Double-Angle Sections Short Legs Verti

ANGLES PLACED BACK TO BACK



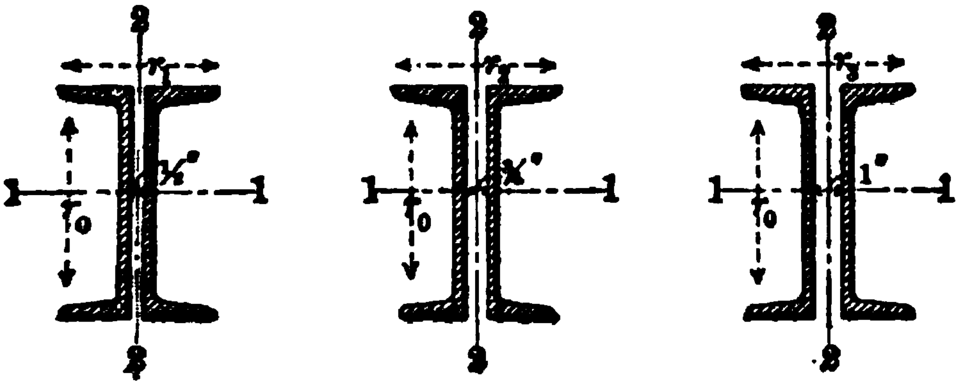


Single angle		Two angles	Radii of gyration, r, in inches				
				Axis 2–2			
Size, in	Weight per foot, lb	Area, sq in	Axis 1–1	In contact	¼-in apart	⅜-in apart	½-in apart
8 × 6 × 1	44.2	26.00	1.73	3.64	3.73	3.78	3.83
¾	33.8	19.88	1.76	3.60	3.69	3.73	3.78
7/16	20.2	11.86	1.80	3.55	3.64	3.68	3.73
8 × 3½ × 1	35.7	21.00	0.86	4.04	4.14	4.19	4.24
¾	27.5	16.12	0.88	3.99	4.09	4.13	4.18
7/16	16.5	9.68	0.92	3.93	4.02	4.07	4.12
7 × 3½ × 1	32.3	19.00	0.89	3.48	3.58	3.63	3.68
11/16	23.0	13.50	0.92	3.42	3.52	3.57	3.62
⅜	13.0	7.60	0.96	3.36	3.46	3.50	3.55
6 × 4 × 1	30.6	18.00	1.09	2.85	2.95	2.99	3.04
11/16	21.8	12.80	1.13	2.79	2.89	2.93	2.98
⅜	12.3	7.22	1.17	2.74	2.83	2.87	2.92
6 × 3½ × 1	28.9	17.00	0.92	2.92	3.02	3.07	3.12
11/16	20.6	12.12	0.95	2.87	2.96	3.01	3.06
5/16	9.8	5.74	1.00	2.81	2.90	2.95	3.00
5 × 4 × ⅞	24.2	14.22	1.14	2.29	2.38	2.43	2.48
⅜	11.0	6.46	1.20	2.20	2.29	2.34	2.38
5 × 3½ × ⅞	22.7	13.34	0.96	2.36	2.45	2.50	2.55
5/16	8.7	5.12	1.03	2.26	2.35	2.39	2.44
5 × 3 × 13/16	19.9	11.68	0.80	2.42	2.52	2.57	2.62
5/16	8.2	4.80	0.85	2.33	2.42	2.47	2.52
4½ × 3 × 13/16	18.5	10.86	0.81	2.15	2.25	2.30	2.35
5/16	7.7	4.50	0.87	2.06	2.15	2.20	2.25
4 × 3½ × 13/16	18.5	10.86	1.01	1.81	1.91	1.96	2.01
5/16	7.7	4.50	1.07	1.73	1.81	1.86	1.91
4 × 3 × 13/16	17.1	10.06	0.83	1.88	1.98	2.03	2.08
¼	5.8	3.38	0.89	1.78	1.87	1.92	1.96
3½ × 3 × 13/16	15.8	9.24	0.85	1.61	1.71	1.76	1.81
¼	5.4	3.12	0.91	1.52	1.61	1.65	1.70
3½ × 2½ × 11/16	12.5	7.30	0.69	1.66	1.75	1.80	1.86
¼	4.9	2.88	0.74	1.58	1.67	1.71	1.76
3 × 2½ × 9/16	9.5	5.56	0.72	1.37	1.46	1.51	1.56
¼	4.5	2.64	0.75	1.31	1.40	1.45	1.50
3 × 2 × ½	7.7	4.50	0.55	1.42	1.52	1.57	1.62
¼	4.1	2.38	0.57	1.38	1.47	1.52	1.57
2½ × 2 × ½	6.8	4.00	0.58	1.15	1.25	1.30	1.35
¼	3.62	2.12	0.59	1.11	1.20	1.25	1.30

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XVIII. Properties of Double-Channel Sections

STANDARD CHANNELS PLACED BACK TO BACK



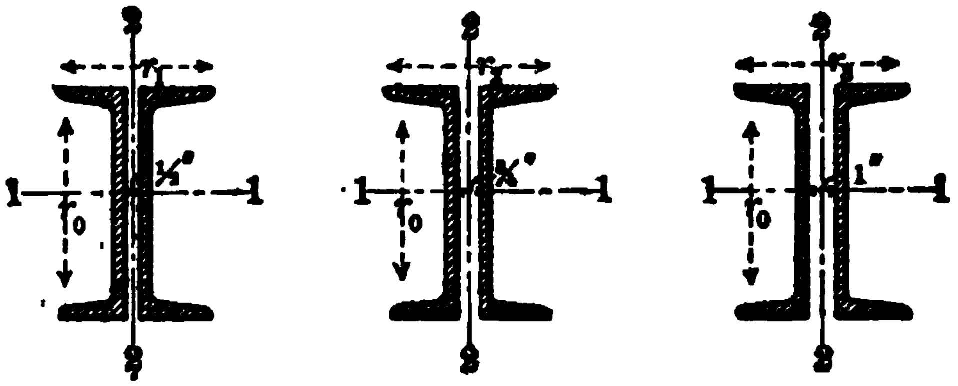
...e radii of gyration given correspond to directions indicated by the arrow-heads

...pth, in	Thickness of web, in	Weight per foot of one channel, lb	Area of two channels, sq in	Radii of gyration, r, in inches			
				Axis 1-1	Axis 2-2 *		
					½-in apart	¾-in apart	1-in apart
	0.40	33.90	19.80	5.62	1.38	1.48	1.58
	0.42	35.00	20.46	5.58	1.38	1.47	1.57
15	0.52	40.00	23.40	5.44	1.37	1.46	1.56
	0.62	45.00	26.34	5.32	1.37	1.45	1.56
	0.72	50.00	29.28	5.24	1.37	1.46	1.56
	0.81	55.00	32.22	5.16	1 38	1.47	1.58
	0.28	20.57	12.06	4.61	1.24	1.34	1.44
	0.39	25.00	14.64	4.43	1.21	1.31	1.41
12	0.51	30.00	17.58	4.28	1.20	1,30	1.40
	0.63	35.00	20.52	4.18	1.21	1.31	1.41
	1.76	40.00	23.46	4.09	1.23	1.32	1.43
	0.24	15.30	8.94	3.87	1.14	1.24	1.34
	0.38	20.00	11.72	3.66	1.10	1.20	1.31
10	0.53	25.00	14.66	3.52	1.10	1.20	1.31
	0.67	30.00	17.60	3.42	1.12	1.22	1.33
	0.82	35.00	20.54	3.34	1.16	1.26	1.37
	0.23	13.40	7.78	3.49	1.09	1.19	1.29
	0.29	15.00	8.78	3.40	1.07	1.17	1.28
9	0.45	20.00	11.72	3.22	1.05	1.15	1.26
	0.61	25.00	14.66	3.10	1.07	1.17	1.28

...re are very slight variations from these values caused by slight changes in weights ...t, but the values are sufficiently exact for all practical uses.

Table XVIII (Continued). Properties of Double-Channel Sections

STANDARD CHANNELS PLACED BACK TO BACK



The radii of gyration given correspond to directions indicated by the arrow

Depth, in	Thickness of web, in	Weight per foot of one channel, lb	Area of two channels, sq in	Radii of gyration, r, in inch		
				Axis 1–1	Axis 2–2 *	
					½-in apart	¾-in apart
8	0.22	11.50	6.72	3.10	1.04	1.14
8	0.30	13.75	8.04	2.99	1.04	1.14
8	0.40	16.25	9.52	2.89	1.03	1.14
8	0.49	18.75	10.98	2.82	1.03	1.14
8	0.58	21.25	12.46	2.77	1.03	1.14
7	0.21	9.75	5.70	2.72	0.99	1.09
7	0.31	12.25	7.16	2.59	0.99	1.09
7	0.42	14.75	8.64	2.51	0.99	1.10
7	0.52	17.25	10.10	2.44	1.00	1.10
7	0.63	19.75	11.58	2.39	1.00	1.10
6	0.20	8.20	4.78	2.34	0.94	1.05
6	0.31	10.50	6.14	2.22	0.94	1.05
6	0.44	13.00	7.62	2.13	0.95	1.06
6	0.56	15.50	9.08	2.07	0.95	1.06
5	0.19	6.70	3.90	1.95	0.89	1.00
5	0.33	9.00	5.26	1.83	0.90	1.00
5	0.47	11.50	6.72	1.76	0.91	1.01
4	0.18	5.40	3.12	1.56	0.84	0.95
4	0.25	6.25	3.64	1.50	0.84	0.95
4	0.32	7.25	4.24	1.47	0.84	0.95
3	0.17	4.10	2.38	1.17	0.80	0.91
3	0.26	5.00	2.92	1.12	0.81	0.92
3	0.36	6.00	3.50	1.08	0.83	0.93

* See note on page 373.

CHAPTER XI

RESISTANCE TO TENSION. PROPERTIES OF IRON AND STEEL

By

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1. Definitions, Working Stresses and Examples

The Ultimate Tensile Strength of a material is the amount of internal stress which a section one square inch in area is capable of exerting against an external axial force. It is the UNIT STRESS OR INTENSITY OF STRESS, expressed in pounds per square inch, which the material can withstand. It is often called the ULTIMATE STRENGTH OR ULTIMATE STRESS of the material. Its value for any material depends on the tenacity of the fibers or the cohesion of the particles of which the material is composed.

An Axial Force is one which acts uniformly over the section of a prismatic body so that the resultant of the distributed forces coincides with the axis of the body. Hence the total AXIAL FORCE which any cross-section of a body will resist is the product of the ultimate strength of the material and the area of the cross-section, in square inches.

Safe Working Stress. The ultimate strength of different building materials has been found by pulling apart bars of known dimensions and dividing the maximum load each sustained by the area of the bar before testing. This ultimate strength, however, must not be used to proportion the size of members in structures, because of variations in material, hidden defects and imperfect workmanship; and, especially, because of indefiniteness as to the maximum load that may be imposed on the structure. To provide safety against the rupture of a member and the consequent failure of the structure from any of these causes, the proportions of the members must be based on SAFE WORKING STRESSES which are usually some fractional part of the ultimate strength found by experiment to provide proper security against failure.

The Factor of Safety is the ratio of the ultimate strength to this safe working stress for that material. Its value ranges generally from 2 to 10, depending upon the nature of the material and the service to which it is applied.

Safe Working Stress in Tension. Table I gives these values for various building materials. The total SAFE LOAD that may be applied to a piece of material of uniform section is found by multiplying the cross-section of the piece, in square inches, by the safe working stress opposite the name of the material of which the piece is composed.

Then if P = the safe load in lb,

S_t = the allowable safe working stress in tension,

b = the width of a rectangular bar,

h = the depth of a rectangular bar,

d = the diameter of a round bar,

it results, for a rectangular bar,

$$P = bhS_t \tag{1}$$

for a round bar,

$$P = 0.7854\, d^2 S_t \tag{2}$$

The area of cross-section to support a load P is, for a rectangular bar,

$$A = \frac{P}{S_t}$$

and for a round bar

$$d = \sqrt{\frac{P}{0.7854\, S_t}}$$

Table I. Safe Working Stress in Tension for Building Materials *

Material	Safe stress lb per sq in (
Cast iron	3 000
Wrought iron	12 000
Steel, medium	16 000
Chestnut	850
Douglas fir	800
Hemlock	600
Pine. long-leaf yellow	1 200
Pine, short-leaf yellow	900
Pine, Norway	800
Pine, white	700
Redwood	700
Spruce	800
White oak	1 200

* **Note.** For woods these values may be increased up to 30% for selected, pe protected, commercially dry timber, not subject to impact, that is, for ideal cond (See, also, pages 637 and 647.)

Example 1. What size of medium-steel angle should be used to sust tensile force of 64 000 lb?

Answer. By Formula (3),

$$\text{the net sectional area} = \frac{64\,000}{16\,000} = 4.00 \text{ sq in}$$

From the Table of the Properties of Angles (Chapter X) we find that a 4 by ⅝-in angle has an area of 4.61 sq in, which is to be reduced by a ⅞-in for a ¾-in rivet, leaving $4.61 - (⅞ \times ⅝) = 4.06$ sq in, net area. This is sl in excess of the required amount.

The SAFE LOAD for angles commonly used in roof-trusses is given in Tal and the REDUCTION IN SECTIONAL AREA caused by rivet-holes, in Tabl this chapter, and in Table I, Chapter XX. See also, Chapter XII, pag paragraph on Punching Rivet-Holes.

Example 2. What size of white-pine tie-beam should be used to sus tensile force of 60 000 lb?

Answer. By Formula (3),

$$\text{the net sectional area} = \frac{60\,000}{700} = 85.7 \text{ sq in}$$

If the depth is taken at 12 in, the net width must be $\frac{85.7}{12} = 7.2$ in. Allo must be made for the increase in tension on the lower side of the beam, its own weight, and also for any cutting that may be necessary in maki connections or holes for truss-rods. If there is a 2-in hole through the b

by 12-in timber must be used. This makes allowance for the weight of the [illegible] itself. If the unsupported length of the beam is great, the allowance for [illegible] weight must be made according to the methods explained in Chapter XV, p. 572, for the calculation of tie-beams subjected to transverse loading.

2. Wrought Iron

Manufacture. Wrought iron is a mixture of pure iron and slag, about [illegible]% iron and 3% slag, together with from ½ to ¾% of other elements including [illegible], phosphorus, sulphur and manganese. It is made from pig iron and [illegible] oxide, or mill-scale, in a reverberatory furnace consisting of a firebox, a [illegible] or working-chamber, and the necessary dampers and flues. The impur[ities] are removed from the iron at different stages in the process, silicon and [man]ganese during the melting-down stage, part of the phosphorus and sulphur [dur]ing the clearing-stage and the carbon and remainder of the phosphorus and [sul]phur during the boiling-stage. The iron is then in a pasty condition ready [for] a thorough stirring by the workman, who collects it into balls of about 80 lb [we]ight and takes it to a squeezer or forge where the greater part of the slag is [remo]ved. It is then rolled out into MUCK-BARS. These bars are cut into pieces [whi]ch are piled into bundles suited to the size of the finished bar. The piles [are] heated and rolled again. The rolling reduces the amount of slag and makes [the] material denser. The process of rerolling may be repeated a number of [tim]es to produce double or triple-refined MERCHANT-BAR IRON.

The Appearance of Wrought Iron is very much like that of steel. It may [be] distinguished from steel by nicking one side of the bar and bending it away [from] the nick. Iron will split along the slag-laminations and show the COARSELY [fibro]us nature of the material; while steel will bend or rupture at the nick [with]out splitting, any fracture being FINELY FIBROUS or CRYSTALLINE. When [brok]en in a tension-test wrought iron shows a dark fibrous fracture. If the [speci]men is grooved before testing or broken in impact the fracture will be [part]ly crystalline.

Welds. Wrought iron is more easily welded than steel because the work [ma]y be accomplished through a wider range of temperature than with steel. [A we]ld may develop the full strength of the bar, but tests on hand-forged welds [thro]ugh tie-bars reported by Kirkaldy gave average values of about 60% of [the st]rength of the bar.

Use. Wrought iron is no longer used for the manufacture of structural [shap]es, such as angles, channels and beams, its use for structural work being [practi]cally limited to bars, rods and bolts. It can be worked more easily than [steel i]n threading-machines; and on this account, unless steel is specified, some [com]panies will furnish truss-rods, bolts, etc., in wrought iron.

Specifications * for Wrought Iron. Wrought iron may be purchased under [the] Specifications of the American Society for Testing Materials.

Material Covered. 1. These specifications cover two classes of wrought-iron [bar]s, as determined by the kind of material used in their manufacture, namely:

Class A, as defined in Section 2 (*b*);
Class B, as defined in Section 2 (*c*);

* [Th]ese Specifications for Wrought-Iron Plates are issued by the Society under the [des]ignation A 42. They were adopted in 1913 and revised in 1918. There are also [A.S.]T.M. Standard Specifications for Staybolt Iron, Refined Wrought-Iron Bars, Iron [an]d Chain, etc.

I. Manufacture

Process. 2. (*a*) All plates shall be rolled from piles entirely free from admixture of steel.

(*b*) Piles for Class A plates shall be made from puddle-bars made wholly pig iron and such scrap as emanates from rolling the plates.

(*c*) Piles for Class B plates shall be made from puddle-bars made wholly pig iron or from a mixture of pig iron and cast-iron scrap, together with wro iron scrap.

II. Physical Properties and Tests

Tension-tests. 3. (*a*) The plates shall conform to the following mini requirements as to tensile properties:

Properties considered	Class A		Class B	
	6 in to 24 in incl, in width	Over 24 in to 90 in incl, in width	6 in to 24 in incl, in width	Over : to 9 inc in wi
Tensile strength, lb per sq in	49 000	48 000	48 000	47 (
Yield-point, lb per sq in	26 000	26 000	26 000	26 (
Elongation in 8 in, per cent	16	12	14	1(

(*b*) The yield-point shall be determined by the drop of the beam of the te machine. The speed of the cross-head of the machine shall not exceed ¾ minute.

Modifications in Elongation. 4. For plates under $\frac{5}{16}$ in in thickn deduction of 1 from the percentages of elongation specified in Section 3 sh made for each decrease of $\frac{1}{16}$ in in thickness below $\frac{5}{16}$ in.

Bend Tests. 5. (*a*) COLD-BEND TESTS. The test-specimen shall cold through 90° without fracture on the outside of the bent portion, as fo For Class A plates, around a pin the diameter of which is equal to 1½ tim thickness of the specimen; and for Class B plates, around a pin the diame which is equal to three times the thickness of the specimen.

(*b*) NICK-BEND TESTS. The test-specimen, when nicked on one sid broken, shall show for Class A plates, a wholly fibrous fracture, and for C plates, not more than 10% of the fractured surface to be crystalline.

Test-specimens. 6. Tension and bend-test specimens shall be take the finished plates and shall be of the full thickness of plates as rolled. longitudinal axis of the specimen shall be parallel to the direction in whi plates are rolled.

Number of Tests. 7. (*a*) One tension, one cold-bend and one nic test shall be made for each variation in thickness of ⅛ in and not less th test for every ten plates as rolled.

(*b*) If any test-specimen fails to conform to the requirements specif reason of an apparent local defect, a retest shall be made. If the retest als the plates represented by such test will be rejected.

III. Finish

Finish. 8. The plates shall be straight, smooth, and free from cinde and holes, injurious flaws, buckles, blisters, seams, and laminations.

IV. Marking

Marking. 9. The plates shall be stamped or otherwise marked as designated the purchaser.

V. Inspection and Rejection

Inspection. 10. (*a*) The inspector representing the purchaser shall have entry, at all times while work on the contract of the purchaser is being per- ed, to all parts of the manufacturer's works which concern the manufacture the plates ordered. (See complete Specifications for Sections 10, 11 and 12.)

3. Cast Iron

Cast Iron has been defined as a saturated solution of carbon in iron, the on-content varying from 1½ to 4% according to the other impurities con- ed. It is hard, brittle, non-malleable and very fluid when melted, so that s well adapted for casting into complex forms.

Manufacture. It is produced in the blast-furnace, which is essentially a nd refractory-lined stack, with a valve-charging device at the top, tuyeres penings in the lower part for the introduction of the air-blast, and a hearth the bottom with a tap-hole for the periodic withdrawal of the iron and slag. e FURNACE-IRON is cast into PIGS about 3 ft long and weighing about 100 lb h. FOUNDRY-CASTINGS are made from PIG IRON and SCRAP melted in a cupola poured into green-sand molds. The charge is made up of different quanti- s of the different grades of pig so as to control the physical properties of the ings, principally through control of the silicon-content.

Appearance. CASTINGS have a gray or white fracture according to the condi- of the contained carbon, the gray fracture indicating graphitic or separated on and the white the combined carbon. GRAY IRON is softer and tougher is specified for ordinary CASTINGS.

Strength. Cast iron does not have a definite ELASTIC LIMIT. A relatively l stress will produce some permanent deformation. Its ULTIMATE TENSILE NGTH varies from 15 000 to 20 000 lb per sq in; and in some iron is as high 0 000 lb per sq in. Its COMPRESSIVE STRENGTH varies over a wide range, 00 lb per sq in being a fair average value.

Defects. Castings are liable to several common DEFECTS the chief of which blow-holes due to the formation of steam from the damp molds, sand-holes to misplaced sand, rough surfaces, cold shuts due to chilling of the iron failure to fill the parts of the mold, shrinkage-cracks due to uneven cooling e castings in parts of different thickness. In cored castings, also, the walls frequently of variable thickness because of the shifting of the cores. This is ially frequent in case of hollow columns cast in a horizontal position. use of these defects and on account of the low ULTIMATE STRENGTH, cast should never be used where it is subjected to any great tensile stress.

Specifications* for Cast Iron. The specifications of the American Society Testing Materials, for GRAY-IRON CASTINGS, include the following require- s:

Unless FURNACE-IRON is specified, all GRAY CASTINGS are understood to be by the CUPOLA-PROCESS.

The SULPHUR-CONTENTS are to be:

For light castings, not over 0.10 per cent.
For medium castings, not over 0.10 per cent.
For heavy castings, not over 0.12 per cent.

* These specifications are issued under the fixed designation A 48. They were adopted 5 and revised in 1918. The complete specification can be obtained from the Society.

3. In dividing castings into LIGHT, MEDIUM and HEAVY classes, the follo standards have been adopted:

Castings having any section less than ½ in thick shall be known as CASTINGS.

Castings in which no section is less than 2 inches thick shall be kno HEAVY CASTINGS.

MEDIUM CASTINGS are those not included in the above classification.

4. TRANSVERSE TEST. The minimum BREAKING STRENGTH of the TRATION-BAR under transverse load shall be:

For light castings, not under 2 500 lb.
For medium castings, not under 2 900 lb.
For heavy castings, not under 3 300 lb.

In no case shall the DEFLECTION be under 0.10 in.

TENSION-TEST. Where specified this shall be:

For light castings, not less than 18 000 lb per sq in.
For medium castings, not less than 21 000 lb per sq in.
For heavy castings, not less than 24 000 lb per sq in.

The specifications give explicit directions for casting the ARBITRATIO which is 1¼ in in diameter and 15 in long. Two of these are cast fo twenty tons of castings. One of each pair must fulfill the requirements t mit acceptance of the castings. The bar is loaded at the middle at a rat will cause a 0.10-in deflection in from twenty to forty seconds. The te test is not recommended.

11. CASTINGS shall be true to pattern, free from cracks, flaws and exc shrinkage. In other respects they shall conform to whatever points sl specially agreed upon.

4. Steel

Steel is a mixture of compounds of iron and carbon with small quant other elements, including manganese, phosphorus, sulphur, silicon, etc. carbon-content controls the hardness and strength of the steel. Les 0.10% of carbon is present in the soft steels, which have most of the teristics of wrought iron; while steel with more than 0.40% carbon is of being tempered, cannot be welded and is very much stronger. Mar acts as a cleanser during the process of manufacture, and increases the ab lity of the steel. Phosphorus and sulphur are harmful in their effects phorus making steel brittle under sudden loading and sulphur making it ho or brittle when heated.

Manufacture. STRUCTURAL STEEL is manufactured by the BESSEM the OPEN-HEARTH PROCESSES. In the first, molten cast iron is charged Bessemer converter, an air-blast is driven through the charge from perfo in the false bottom of the converter and the silicon, sulphur and carbon out. Carbon in the form of ferro-manganese is then added to deoxic charge and give the proper content of carbon in the finished steel, w quickly drawn off and poured into ingots. Phosphorus is not remove narily by the Bessemer process; but if the lining of the converter is basic material, such as dolomite limestone, and if lime is added with the the phosphorus will unite with it and be poured off with the slag.

The Open-Hearth Process. In this process scrap-steel, pig-iron or furnace-iron and limestone flux are charged on the hearth of a Siemens a reducing gas-flame is directed onto the charge and the carbon an impurities are gradually removed. When the reduction is about complet

are taken and carbon determined so that the charge may be withdrawn at the proper time. The process thus permits of much more accurate control of the product. The material is more uniform and consequently more dependable in service than BESSEMER STEEL. OPEN-HEARTH STEEL is used for most structural work.

Phosphorus may be removed by the basic process as in case of Bessemer steel. Ores running low in phosphorus are generally used in America so that the basic process is little employed here.

The Effect of Carbon and Phosphorus on the STATIC STRENGTH of steel within the limits of carbon included in structural steel is an increase in strength of about 1 000 lb per sq in for each 0.01% increase in either element. Cunningham's formula

$$S_t = 40\,000 + 100\,000\ (C + P)$$

is the approximate relation between the strength and the chemical composition. C and P are respectively the amounts of carbon and phosphorus expressed in percentage. For example, the ULTIMATE STRENGTH of a steel having 0.15% carbon and 0.07% phosphorus is, approximately,

$$S_t = 40\,000 + 100\,000\ (0.15 + 0.07) = 62\,000 \text{ lb per sq in}$$

The Percentage of Elongation decreases as the carbon-content and ULTIMATE STRENGTH increase. An approximate relation is

$$\text{percentage of elongation} = \frac{1\,400\,000}{S_t}$$

Since the TOTAL ELONGATION of a ruptured specimen is due to the local stretching at the point of rupture and the uniform elongation over the whole gauge-length, it is necessary to report the gauge-length when reporting this result. Since the LOCAL ELONGATION is the same for a 2 or an 8-in length, the PERCENTAGE OF ELONGATION for the same material, tested on a 2-in gauge-length, is greater than if measured on an 8-in length.

The Elastic Behavior of a specimen of steel loaded to rupture is best shown by a STRESS-STRAIN DIAGRAM on which the stresses are plotted as vertical ordinates and the elongations or strains as abscissas, as in Fig. 1. Five significant points are shown:

(1) **The Modulus of Elasticity** (E). The relation between the stress and the strain or elongation is called the MODULUS OF ELASTICITY. It is equal to the unit stress divided by the unit strain or deformation and is represented graphically by the tangent of the angle of the initial line with the horizontal. Its value for steel for tension is about 30 000 000 lb per sq in.

(2) **The Elastic Limit** ($E.L.$) is that unit stress beyond which the ratio of stress to strain ceases to be constant, or beyond which the curve ceases to be a straight line.

(3) **The Yield-Point** ($Y.P.$), slightly above or beyond the ELASTIC LIMIT, is the unit stress at which the specimen begins to stretch without increase in load. This stress may be determined from a test without the use of delicate measuring-apparatus by the DROP OF THE BEAM OF HALT IN THE GAUGE of the testing-machine.

(4) **The Ultimate Strength** ($U.S.$) is the greatest unit stress the specimen can sustain.

(5) **The Rupture-Stress** (R) is the unit stress at the time of failure. This is the unit stress at the point of failure after the area of the cross-section of the

specimen has been reduced; and because of the rapid dropping off of the l
it is difficult to determine. It is not regularly observed in testing, attent
being called to it merely to emphasize the fact that the ULTIMATE STRENGT
steel is not the stress at the time of failure of the specimen. This is true, a
for wrought iron and ductile materials in general.

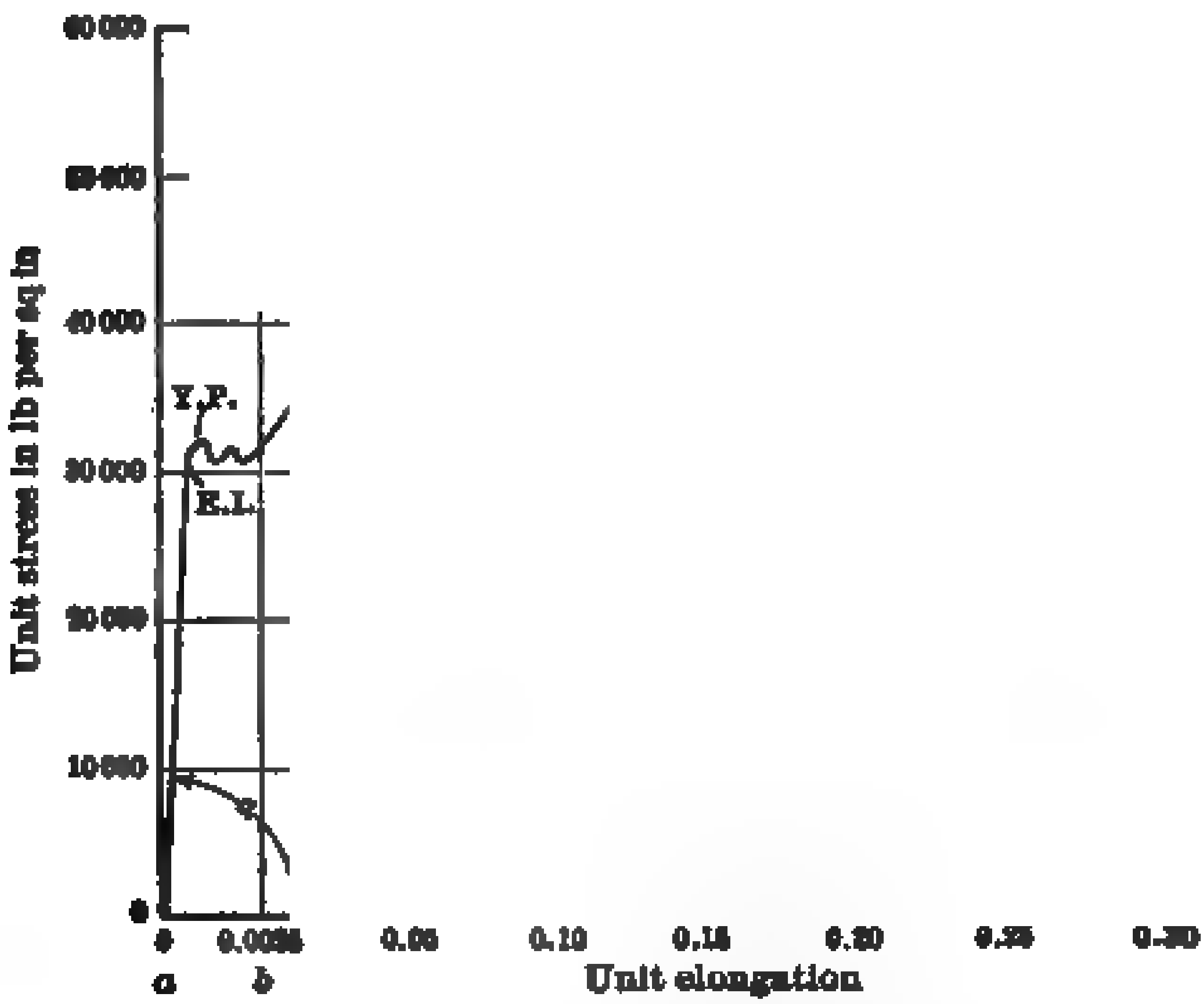


Fig. 1. Stress-strain Diagram of Test on Steel Specimens.

Effect of Punching and Shearing. Structural steel is hardened by
action of the punch and shear in the process of manufacture in the shop.
the die-side the metal is forced to flow from the tool and this cold wo
hardens and injures it as may be shown by a cold-bend test. The effect
be removed by annealing; but in the best work it is usually specified
rivet-holes shall be reamed during the assembling of the parts. This rem
the injured metal and brings the parts into better alinement for the ins
of the rivets. The injury from shearing may be removed by milling the sh
edges.

The Coefficient of Expansion of steel is 0.000 006 5 per degree Fahre
The ELONGATION in a length l, due to a change in temperature of t degr
then

$$e = 0.000\,006\,5\,lt$$

in which l is expressed in inches and t in degrees Fahrenheit.

The Weight of Steel is taken at 489.6 lb per cu ft. The sectional a
a member in square inches multiplied by 3.4 equals the weight in pounds
linear foot.

The Working Stress for structural steel in tension in buildings and bri

o lb per sq in in most specifications and building laws. For members ct to constant load some designers use a WORKING STRESS of 20 000 lb q in.

6. Standard Specifications for Structural Steel for Buildings

cifications. These specifications are issued by the American Society for g Materials under the fixed designation A 9. They were adopted in 1901 vised in 1909, 1913, 1914 and 1916. Extracts from these specifications :

I. Manufacture

cess. 1. (*a*) Structural steel, except as noted in Paragraph (*b*), may be by the Bessemer or the open-hearth process.

Rivet steel, and steel for plates or angles over ¾ in in thickness which are nched, shall be made by the open-hearth process.

II. Chemical Properties and Tests

mical Composition. 2. The steel shall conform to the following re- nts as to chemical composition:

Chemical content	Structural steel	Rivet steel
phorus { Bessemer......	not over 0.10 per cent	....
phorus { open-hearth....	not over 0.06 per cent	not over 0.06 per cent
hur..................	....	not over 0.045 per cent

le Analyses. 3. An analysis of each melt of steel shall be made by the cturer to determine the percentages of carbon, manganese, phosphorus lphur. This analysis shall be made from a test-ingot taken during the g of the melt. The chemical composition thus determined shall be re- l to the purchaser or his representative, and shall conform to the require- specified in Section 2.

ck Analyses. 4. Analyses may be made by the purchaser from finished al representing each melt. The phosphorus and sulphur-content thus ined shall not exceed that specified in Section 2 by more than 25 per cent.

III. Physical Properties and Tests

sion-Tests. 5. (*a*) The material shall conform to the following require- as to tensile properties:

Properties considered	Structural steel	Rivet steel
e strength, lb per sq in............	55 000–65 000	46 000–56 000
point, min, lb per sq in............	0.5 tens. strength	0.5 tens. strength
ation in 8 in, min, per cent.........	$\frac{1\,400\,000^*}{\text{tens. strength}}$	$\frac{1\,400\,000}{\text{tens. strength}}$
ation in 2 in, min, per cent.........	22	......

* See Section 6.

The yield-point shall be determined by the drop of the beam of the testing- e.

Modifications in Elongation. 6. (*a*) For structural steel over ¾ in in
ness, a deduction of 1 from the percentage of elongation in 8 in, specif
Section 5 (*a*), shall be made for each increase of ⅛ in in thickness above
to a minimum of 18 per cent.

(*b*) For structural steel under 5/16 in in thickness, a deduction of 2.5 fr
percentage of elongation in 8 in, specified in Section 5 (*a*), shall be made fo
decrease of 1/16 in in thickness below 5/16 in.

Bend Tests. 7. (*a*) The test-specimen for plates, shapes and bars, exc
specified in Paragraphs (*b*) and (*c*), shall bend cold through 180° without cr
on the outside of the bent portion, as follows: For material ¾ in or un
thickness, flat on itself; for material over ¾ in, to and including 1¼ in in
ness, around a pin the diameter of which is equal to the thickness of the spe
and for material over 1¼ in in thickness, around a pin the diameter of w
equal to twice the thickness of the specimen.

(*b*) The test-specimen for pins, rollers, and other bars, when prepa
specified in Section 8 (*e*), shall bend cold through 180° around a 1-in pin w
cracking on the outside of the bent portion.

(*c*) The test-specimen for rivet steel shall bend cold through 180° flat o
without cracking on the outside of the bent portion.

Test-Specimens. 8. (*a*) Tension and bend-specimens shall be take
rolled steel in the condi
which it comes from th
except as specified in
graph (*b*).

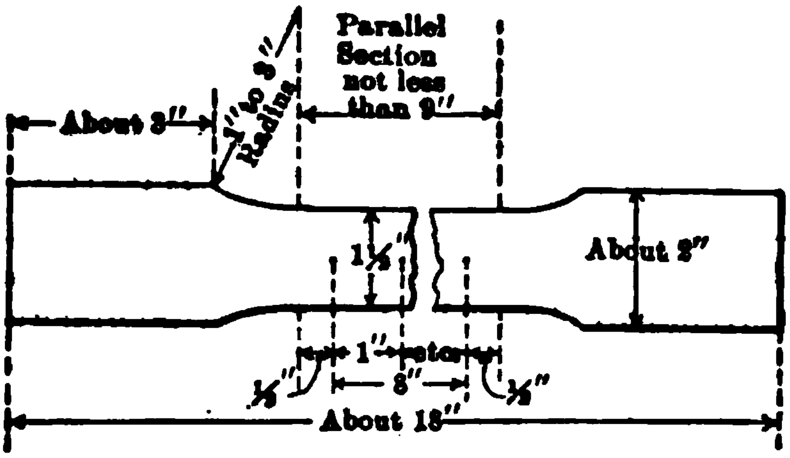


Fig. 1A. Form of Specimen for Steel-test

(*b*) Tension and be
specimens for pins and
shall be taken from t
ished bars after ann
when annealing is speci

(*c*) Tension and be
specimens for plates,
and bars, except as sp
in Paragraphs (*d*), (*e*
(*f*), shall be of the full thickness of the material as rolled; and s
machined to the form and dimensions shown in Fig. 1A, or with both
parallel.

(*d*) Tension and bend-test specimens for plates over 1½ in in thickness may be machined to a thickness or diameter of at least ¾ in for a length of at least 9 in.

(*e*) Tension-test specimens for pins, rollers and bars over 1½ in in thickness or diameter may conform to the dimensions shown in Fig. 2. In this case, the ends shall be of a form to fit the holders of the

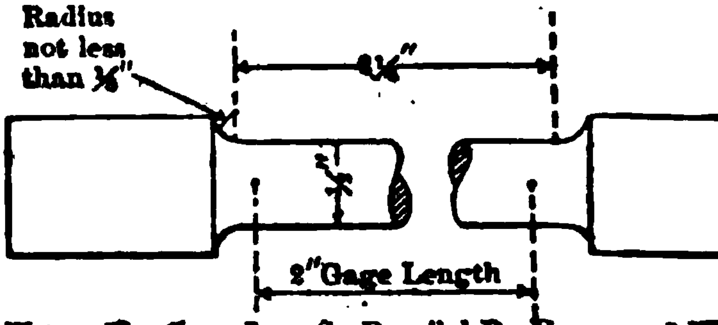


Fig. 2. Form of Specimen for Pins, Rollers,
etc., Over 1½ Inches Thick

testing-machine in such a way that the load shall be the axial. Bend-tes
mens may be 1 by ½ in in section. The axis of the specimen shall be lo
any point midway between the center and surface and shall be paralle
axis of the bar.

Tension and bend-test specimens for rivet steel shall be of the full-size n of bars as rolled.

mber of Tests. 9. (*a*) One tension and one bend-test shall be made from melt; except that if material from one melt differs ⅜ in or more in thickness, nsion and one bend-test shall be made for both the thickest and the st material rolled.

If any test-specimen shows defective machining or develops flaws, it may rded and another specimen substituted.

If the percentage of elongation of any tension-test specimen is less than ecified in Section 5 (*a*) and any part of the fracture is more than ¾ in the center of the gauge-length of a 2-in specimen or is outside the middle of the gauge-length of an 8-in specimen, as indicated by scribe-scratches ed on the specimen before testing, a retest shall be allowed.

IV. Permissible Variations in Weight and Thickness

rmissible Variations. 10. The cross-section or weight of each piece of steel not vary more than 2.5 per cent from that specified; except in case of ed plates, which shall be covered by the following permissible variations. ubic inch of rolled steel is assumed to weigh 0.2833 lb.

When Ordered to Weight per Square Foot: The weight of each lot in hipment shall not vary from the weight ordered more than the amount in Table I.*

When Ordered to Thickness: The thickness of each plate shall not more than 0.01 in under that order.

e overweight of each lot in each shipment shall not exceed the amount in Table II.*

V. Finish

ish. 11. The finished material shall be free from injurious defects and shall s workmanlike finish.

VI. Marking

rking. 12. The name or brand of the manufacturer and the melt-number be legibly stamped or rolled on all finished material, except that rivet and e-bars and other small sections shall, when loaded for shipment, be properly ted and marked for identification. The identification-marks shall be y stamped on the end of each pin and roller. The melt-number shall be y marked, by stamping if practical, on each test-specimen.

VII. Inspection and Rejection

pection. 13. (See complete Specifications for Sections 13, 14 and 15.)

6. Tension-Members

ges. The best section for tension-members of relatively small size depends y on the kind of end-connections used. Angles or channels are generally r riveted connections. For very small members rectangular bars, such as bars, may be used. The strength of such members is computed on the net hrough the rivet-holes. Angles used in tension should have lugs riveted e outstanding legs and the tie-plate for the better distribution of the stress the section. Tests on angles with riveted connections reported by F. Kibbent† gave from 77 to 86% of the strength of the material as shown by

* es I and II are omitted here for lack of space. The complete specifications can ined from the Society.

† ceedings of the American Society for Testing Materials, Vol. VI, 1906.

tension-tests on standard specimens cut from these angles. Lugs increase strength from 4.7 to 8.7%. It was also shown that a connection giving center of the pull on the center of gravity of the section gave considerably hi strengths than when the center of pull was in line with the gauge-line o rivets. In computing the NET SECTIONAL AREA as reduced by rivet and holes Table XI will be found very convenient.

Eye-Bars are used for the main tension-members of pin-connected tr They are rectangular in section with a forged head upset in dies and of the thickness as the bar. The eye is accurately drilled in position in the axis o bar, true to diameter and exact central distance. Because of its advan for forging, soft steel is used in making eye-bars. They are also carefully nealed before drilling. Table VI gives the dimensions of STANDARD EYE- manufactured by the mills of the American Bridge Company. These are of practically the same dimensions as the standard bars of other panies. There is from 34 to 42% excess material in the section through th to insure in the forged part the development of the full strength of the bo the bar. Standard bars should be used in design to avoid the expense of m special dies in which to form the heads. Bars of less than the given mini thickness are liable to fail, when loaded, by buckling in the head. Thick increase the BENDING-STRESSES in the pins and thus, indirectly, the nec size of the eye. Except for very large structures they are limited to 2 in.

Tests of Full-Size Eye-Bars are generally required when a great num them are to be used in a structure, one in every fifty bars being usually t The specifications for carbon-steel bars require that an ULTIMATE TE STRENGTH of 56 000 lb per sq in shall be developed, that the ELONGATION whole length shall be 10% and that failure shall occur in the body of th Nickel steel has been used for tension-members on a few long-span bridges. WORKING STRESS on the eye-bars was increased about one-half over that us

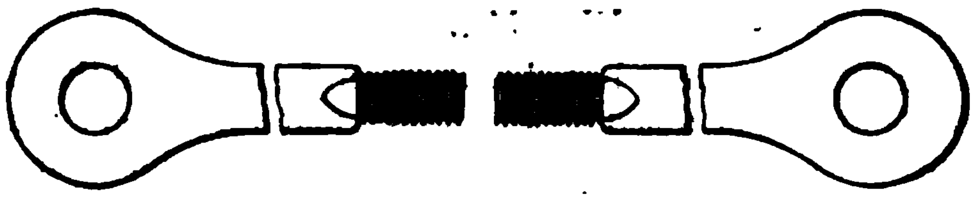


Fig. 3. Eye-bar with Screw-ends for Sleeve-nut or Turn-buckle

carbon steel, and the requirements of the test-bars made correspondingly The eye is made 1/50 in greater than the diameter of the pin. Bars pac the same pins are drilled at the same setting so as to be of exactly th length. Bars must be true to length within 1/32 in. Small eye-bars are times made with UPSET SCREW-ENDS and SLEEVE-NUTS or TURNBUCKLES middle for adjustment, as shown in Fig. 3 and Table VI, page 395.

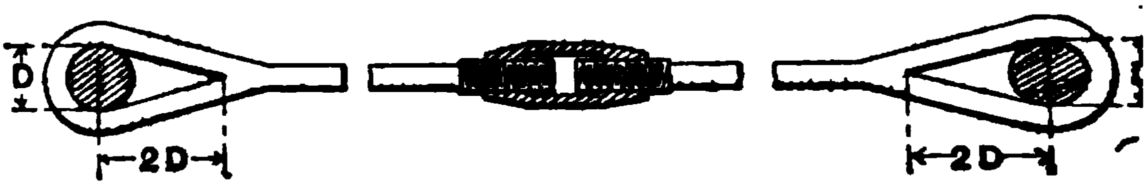


Fig. 4. Loop-eyes and Sleeve-nuts

Loop-Rods (Fig. 4, and Table VII) of round or square section with loop-ends are used for counterties and bracing. Because of the wel are not so dependable as other types of tension-members, but, because adjustment, are well adapted for this service as secondary members.

Forked-Loop Rod, Fig. 5, may be used for one of two tension-rods so as aid eccentricity where two rods balance each other on a pin. A CLEVIS ach end of one of the rods mplishes the same object.

mbuckles and Sleeve-Nuts. dimensions of these for adjust-ble lengths and initial stress in are given in Table VIII, page The open turnbuckle has the ntage of being easily inspected ite that the thread has sufficient ng and that the ends of the do not butt together.

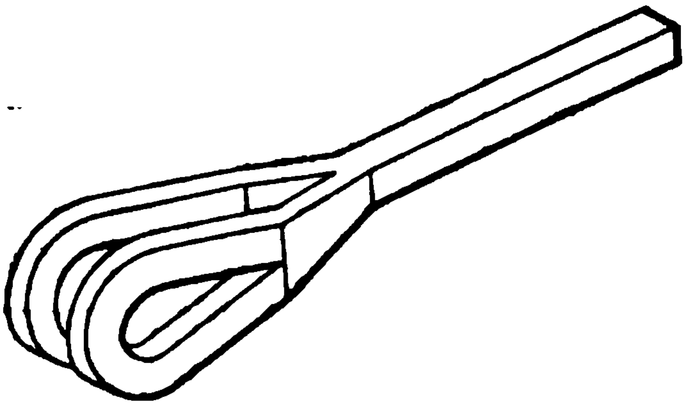


Fig. 5. Forked Loop

pset Screw-Ends are threaded enlargements on the ends of rods or bolts ed to give to the threaded portions a strength as great as that of the body e bar. Because of effects of forging it is necessary to make the area of the section of the upset end at the root of the thread a little larger than that e rod itself. A standard upset rod will fail in the body of the bar with-amaging the threaded portion enough to prevent the turning of the nuts. dimensions given are nearly the same with all manufacturers. If upset can not be obtained the section-area at the root of the thread must be in computing the safe load.

evises. Table IX, page 398, gives the dimensions and other details for es according to the latest standards of the American Bridge Company.

bles. The following tables will be found useful in designing tension-bers, or for drawing turnbuckles, sleeve-nuts, clevises, etc. The strength in rods in Table II is based on the area at the root of the thread. For s and weights of tie-rods and anchors for steel beams, see Table XIX, ter XV.

Table II. Safe Loads in Pounds on Round Rods

Diameter inches	Plain rods Load in pounds based on area at root of thread			Upset rods Load in pounds based on f area of rod		
	Stress in lb per sq in			Stress in lb per sq in		
	10 000	12 000	16 000	10 000	12 000	16
¼	270	324	432	491	590	
5/16	450	540	720	767	920	1
⅜	680	816	1 088	1 104	1 320	1
7/16	930	1 116	1 488	1 503	1 800	2
½	1 260	1 513	2 016	1 963	2 360	3
9/16	1 620	1 944	2 592	2 485	2 970	3
⅝	2 020	2 424	3 232	3 068	3 680	4
¾	3 020	3 624	4 832	4 418	5 300	7
⅞	4 200	5 040	6 720	6 013	7 210	9
1	5 500	6 600	8 800	7 854	9 420	12
1⅛	6 940	8 328	11 104	9 940	11 930	15
1¼	8 930	10 716	14 288	12 270	14 720	19
1⅜	10 570	12 680	16 910	14 840	17 810	23
1½	12 950	15 540	20 720	17 670	21 200	28
1⅝	15 150	18 180	24 240	20 730	24 880	33
1¾	17 440	20 930	27 900	24 050	28 860	38
1⅞	20 480	24 580	32 760	27 610	33 130	44
2	23 020	27 620	36 830	31 420	37 700	50
2⅛	26 340	31 610	42 150	35 460	42 550	56
2¼	30 230	36 280	48 370	39 760	47 710	63
2⅜	33 000	39 600	52 800	44 300	53 160	70
2½	37 150	44 630	59 440	49 080	58 900	78
2¾	46 190	55 430	73 900	59 390	71 270	9
3	54 280	65 140	86 850	70 680	84 820	11
3¼	65 100	78 120	104 160	82 950	99 540	13
3½	75 480	90 570	120 770	96 210	115 450	15
3¾	86 410	103 690	138 250	110 450	132 540	17
4	99 930	119 920	159 890	125 660	150 790	20
4¼	113 290	135 900	181 300	141 800	170 160	22
4½	127 430	152 900	203 900	159 000	190 800	25
4¾	142 200	170 600	227 500	177 200	212 640	28
5	157 630	189 100	252 200	196 300	235 560	31
5¼	175 720	210 800	281 100	216 400	259 680	34
5½	192 670	231 200	308 300	237 500	285 000	38
5¾	212 620	255 100	340 200	259 600	311 000	41
6	230 980	277 200	369 600	282 700	339 200	45

Table III. Safe Loads in Pounds for Flat Rolled Bars

Computed for a stress of 16 000 pounds per square inch

Thickness in inches	Width in inches									
	1	1¼	1½	1¾	2	2¼	2½	2¾	3	3¼
1/16	1 000	1 250	1 500	1 750	2 000	2 250	2 500	2 750	3 000	3 250
1/8	2 000	2 500	3 000	3 500	4 000	4 500	5 000	5 500	6 000	6 500
3/16	3 000	3 750	4 500	5 250	6 000	6 750	7 500	8 250	9 000	9 750
1/4	4 000	5 000	6 000	7 000	8 000	9 000	10 000	11 000	12 000	13 000
5/16	5 000	6 250	7 500	8 750	10 000	11 250	12 500	13 750	15 000	16 250
3/8	6 000	7 500	9 000	10 500	12 000	13 500	15 000	16 500	18 000	19 500
7/16	7 000	8 750	10 500	12 250	14 000	15 750	17 500	19 250	21 000	22 750
1/2	8 000	10 000	12 000	14 000	16 000	18 000	20 000	22 000	24 000	26 000
9/16	9 000	11 250	13 500	15 750	18 000	20 250	22 500	24 750	27 000	29 250
5/8	10 000	12 500	15 000	17 500	20 000	22 500	25 000	27 500	30 000	32 500
11/16	11 000	13 750	16 500	19 250	22 000	24 750	27 500	30 250	33 000	35 750
3/4	12 000	15 000	18 000	21 000	24 000	27 000	30 000	33 000	36 000	39 000
13/16	13 000	16 250	19 500	22 750	26 000	29 250	32 500	35 750	39 000	42 250
7/8	14 000	17 500	21 000	24 500	28 000	31 500	35 000	38 500	42 000	45 500
15/16	15 000	18 750	22 500	26 250	30 000	33 750	37 500	41 250	45 000	48 750
1	16 000	20 000	24 000	28 000	32 000	36 000	40 000	44 000	48 000	52 000
1 1/16	17 000	21 250	25 500	29 750	34 000	38 250	42 500	46 750	51 000	55 250
1 1/8	18 000	22 500	27 000	31 500	36 000	40 500	45 000	49 500	54 000	58 500
1 3/16	19 000	23 750	28 500	33 250	38 000	42 750	47 500	52 250	57 000	61 750
1 1/4	20 000	25 000	30 000	35 000	40 000	45 000	50 000	55 000	60 000	65 000
1 3/8	22 000	27 500	33 000	38 500	44 000	49 500	55 000	60 500	66 000	71 500
1 1/2	24 000	30 000	36 000	42 000	48 000	54 000	60 000	66 000	72 000	78 000
1 5/8	26 000	32 500	39 000	45 500	52 000	58 500	65 000	71 500	78 000	84 500
1 3/4	28 000	35 000	42 000	49 000	56 000	63 000	70 000	77 000	84 000	91 000
1 7/8	30 000	37 500	45 000	53 500	60 000	67 500	75 000	83 500	90 000	97 500
2	32 000	40 000	48 000	56 000	64 000	72 000	80 000	88 000	96 000	104 000

Table III (Continued). Safe Loads in Pounds for Flat Rolled Bars

Computed for a stress of 16 000 pounds per square inch

Thickness in inches	Width in inches								
	3½	3¾	4	4¼	4½	4¾	5	5½	6
1/16	3 500	3 750	4 000	4 250	4 500	4 750	5 000	5 500	6 000
⅛	7 000	7 500	8 000	8 500	9 000	9 500	10 000	11 000	12 000
3/16	10 500	11 250	12 000	12 750	13 500	14 250	15 000	16 500	18 000
¼	14 000	15 000	16 000	17 000	18 000	19 000	20 000	22 000	24 000
5/16	17 500	18 750	20 000	21 250	22 500	23 750	25 000	27 500	30 000
⅜	21 000	22 500	24 000	25 500	27 000	28 500	30 000	33 000	36 000
7/16	24 500	26 250	28 000	29 750	31 500	33 250	35 000	38 500	42 000
½	28 000	30 000	32 000	34 000	36 000	38 000	40 000	44 000	48 000
9/16	31 500	33 750	36 000	38 250	40 500	42 750	45 000	49 500	54 000
⅝	35 000	37 500	40 000	42 500	45 000	47 500	50 000	55 000	60 000
11/16	38 500	41 250	44 000	46 750	49 500	52 250	55 000	60 500	66 000
¾	42 000	45 000	48 000	51 000	54 000	57 000	60 000	66 000	72 000
13/16	45 500	48 750	52 000	55 250	58 500	61 750	65 000	71 500	78 000
⅞	49 000	52 500	56 000	59 500	63 000	66 500	70 000	77 000	84 000
15/16	52 500	56 250	60 000	63 750	67 500	71 250	75 000	82 500	90 000
1	56 000	60 000	64 000	68 000	72 000	76 000	80 000	88 000	96 000
1 1/16	59 500	63 750	68 000	72 250	76 500	80 750	85 000	93 500	102 000
1⅛	63 000	67 500	72 000	76 500	81 000	85 500	90 000	99 000	108 000
1 3/16	66 500	71 250	76 000	80 750	85 500	90 250	95 000	104 500	114 000
1¼	70 000	75 000	80 000	85 000	90 000	95 000	100 000	110 000	120 000
1⅜	77 000	82 500	88 000	93 500	99 000	104 500	110 000	121 000	132 000
1½	84 000	90 000	96 000	102 000	108 000	114 000	120 000	132 000	144 000
1⅝	91 000	97 500	104 000	110 500	117 000	123 500	130 000	143 000	156 000
1¾	98 000	105 000	112 000	119 000	126 000	133 000	140 000	154 000	168 000
1⅞	105 000	112 500	120 000	127 500	135 000	142 500	150 000	165 000	180 000
2	112 000	120 000	128 000	136 000	144 000	152 000	160 000	176 000	192 000

Table IV. Safe Loads in Pounds for Flat Rolled Bars

Computed for a stress of 10 000 lb per square inch*

Thickness in inches	Width in inches									
	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	2 3/4	3	3 1/4
1/16	630	780	940	1 090	1 250	1 410	1 560	1 720	1 880	2 030
1/8	1 250	1 560	1 880	2 190	2 500	2 810	3 130	3 440	3 750	4 060
3/16	1 880	2 340	2 810	3 280	3 750	4 220	4 690	5 160	5 630	6 090
1/4	2 500	3 130	3 750	4 380	5 000	5 630	6 250	6 880	7 500	8 130
5/16	3 130	3 910	4 690	5 470	6 250	7 030	7 810	8 590	9 380	10 200
3/8	3 750	4 690	5 630	6 560	7 500	8 440	9 380	10 300	11 300	12 200
7/16	4 380	5 470	6 560	7 660	8 750	9 840	10 900	12 000	13 100	14 200
1/2	5 000	6 250	7 500	8 750	10 000	11 300	12 500	13 800	15 000	16 300
9/16	5 630	7 030	8 440	9 840	11 300	12 700	14 100	15 500	16 900	18 300
5/8	6 250	7 810	9 380	10 900	12 500	14 100	15 600	17 200	18 800	20 300
11/16	6 880	8 590	10 300	12 000	13 800	15 500	17 200	18 900	20 600	22 300
3/4	7 500	9 380	11 300	13 100	15 000	16 900	18 800	20 600	22 500	24 400
13/16	8 130	10 200	12 200	14 200	16 300	18 300	20 300	22 300	24 400	26 400
7/8	8 750	10 900	13 100	15 300	17 500	19 700	21 900	24 100	26 300	28 400
15/16	9 380	11 700	14 100	16 400	18 800	21 100	23 400	25 800	28 100	30 500
1	10 000	12 500	15 000	17 500	20 000	22 500	25 000	27 500	30 000	32 500
1 1/16	10 600	13 300	15 900	18 600	21 300	23 900	26 600	29 200	31 900	34 500
1 1/8	11 300	14 100	16 900	19 700	22 500	25 300	28 100	30 900	33 800	36 600
1 3/16	11 900	14 800	17 800	20 800	23 800	26 700	29 700	32 700	35 600	38 600
1 1/4	12 500	15 600	18 800	21 900	25 000	28 100	31 300	34 400	37 500	40 600
1 3/8	13 800	17 200	20 600	24 100	27 500	30 900	34 400	37 800	41 300	44 700
1 1/2	15 000	18 800	22 500	26 300	30 000	33 800	37 500	41 300	45 000	48 800
1 5/8	16 300	20 300	24 400	28 400	32 500	36 600	40 600	44 700	48 800	52 800
1 3/4	17 500	21 900	26 300	30 600	35 000	39 400	43 800	48 100	52 500	56 900
1 7/8	18 800	23 400	28 100	32 800	37 500	42 200	46 900	51 600	56 300	60 900
2	20 000	25 000	30 000	35 000	40 000	45 000	50 000	55 000	60 000	65 000

* For unit stresses of 12 000, 12 500, and 15 000 lb increase by 1/5, 1/4, and 1/2 respectively.

For working strength of wrought iron and steel, see pages 376 and 382.

Table IV (Continued). Safe Loads in Pounds for Flat Rolled Bars

Computed for a stress of 10 000 lb per square inch

Thickness in inches	Width in inches									
	3¼	3¾	4	4¼	4½	4¾	5	5½	6	6½
1/16	2 190	2 340	2 500	2 660	2 810	2 970	3 130	3 440	3 750	4 0
⅛	4 380	4 690	5 000	5 310	5 630	5 940	6 250	6 880	7 500	8 1
3/16	6 560	7 030	7 500	7 970	8 440	8 910	9 380	10 300	11 300	12 2
¼	8 750	9 380	10 000	10 600	11 300	11 900	12 500	13 800	15 000	16 3
5/16	10 900	11 700	12 500	13 300	14 100	14 800	15 600	17 200	18 800	20 3
⅜	13 100	14 100	15 000	15 900	16 900	17 800	18 800	20 600	22 500	24 4
7/16	15 300	16 400	17 500	18 600	19 700	20 800	21 900	24 100	26 300	28 4
½	17 500	18 800	20 000	21 300	22 500	23 800	25 000	27 500	30 000	32 5
9/16	19 700	21 100	22 500	23 900	25 300	26 700	28 100	30 900	33 800	36 6
⅝	21 900	23 400	25 000	26 600	28 100	29 700	31 300	34 400	37 500	40 6
11/16	24 100	25 800	27 500	29 200	30 900	32 700	34 400	37 800	41 300	44 7
¾	26 300	28 100	30 000	31 900	33 800	35 600	37 500	41 300	45 000	48 8
13/16	28 400	30 500	32 500	34 500	36 600	38 600	40 600	44 700	48 800	52 8
⅞	30 600	32 800	35 000	37 200	39 400	41 600	43 800	48 100	52 500	56 9
15/16	32 800	35 200	37 500	39 800	42 200	44 500	46 900	51 600	56 300	60 9
1	35 000	37 500	40 000	42 500	45 000	47 500	50 000	55 000	60 000	65 0
1 1/16	37 200	39 800	42 500	45 200	47 800	50 500	53 100	58 400	63 800	69 1
1⅛	39 400	42 200	45 000	47 800	50 600	53 400	56 300	61 900	67 500	73 1
1 3/16	41 600	44 500	47 500	50 500	53 400	56 400	59 400	65 300	71 300	77 2
1¼	43 800	46 900	50 000	53 100	56 300	59 400	62 500	68 800	75 000	81 3
1⅜	48 100	51 600	55 000	58 400	61 900	65 300	68 800	75 600	82 500	89 4
1½	52 500	56 300	60 000	63 800	67 500	71 300	75 000	82 500	90 000	97 5
1⅝	56 900	60 900	65 000	69 100	73 100	77 200	81 300	89 400	97 500	105 6
1¾	61 300	65 600	70 000	74 400	78 800	83 100	87 500	96 300	105 000	113 8
1⅞	65 600	70 300	75 000	79 700	84 400	89 100	93 800	103 100	112 500	121 [illegible]
2	70 000	75 000	80 000	85 000	90 000	95 000	100 000	110 000	120 000	130 [illegible]

* See foot-note, preceding table.

Table V. Standard Proportions of Upset Screw-Ends for Round and Square Bars

Diam. of round or side of square bar in	Round bars: Diam. of upset screw-end in	Round bars: Diam. of screw at root of thread in	Round bars: Number of threads per inch	Round bars: Excess of effective area of screw-end over bar %	Square bars: Diam. of upset screw-end in	Square bars: Diam. of screw at root of thread in	Square bars: Number of threads per inch	Square bars: Excess of effective area of screw-end over bar %
1/2	3/4	0.620	10	54	3/4	0.620	10	21
9/16	3/4	0.620	10	21	7/8	0.731	9	33
5/8	7/8	0.731	9	37	1	0.837	8	41
11/16	1	0.837	8	48	1	0.837	8	17
3/4	1	0.837	8	25	1 1/8	0.940	7	23
13/16	1 1/8	0.940	7	34	1 1/4	1.065	7	35
7/8	1 1/4	1.065	7	48	1 3/8	1.160	6	38
15/16	1 1/4	1.065	7	29	1 3/8	1.160	6	20
1	1 3/8	1.160	6	35	1 1/2	1.284	6	29
1 1/16	1 3/8	1.160	6	19	1 5/8	1.389	5 1/2	34
1 1/8	1 1/2	1.284	6	30	1 5/8	1.389	5 1/2	20
1 3/16	1 1/2	1.284	6	17	1 3/4	1.490	5	24
1 1/4	1 5/8	1.389	5 1/2	23	1 7/8	1.615	5	31
1 5/16	1 3/4	1.490	5	29	1 7/8	1.615	5	19
1 3/8	1 3/4	1.490	5	18	2	1.712	4 1/2	22
1 7/16	1 7/8	1.615	5	26	2 1/8	1.837	4 1/2	28
1 1/2	2	1.712	4 1/2	30	2 1/8	1.837	4 1/2	18
1 9/16	2	1.712	4 1/2	20	2 1/4	1.962	4 1/2	24
1 5/8	2 1/8	1.837	4 1/2	28	2 3/8	2.087	4 1/2	30
1 11/16	2 1/8	1.837	4 1/2	18	2 3/8	2.087	4 1/2	20
1 3/4	2 1/4	1.962	4 1/2	26	2 1/2	2.175	4	21
1 13/16	2 1/4	1.962	4 1/2	17	2 5/8	2.300	4	26
1 7/8	2 3/8	2.087	4 1/2	24	2 5/8	2.300	4	18
1 15/16	2 1/2	2.175	4	26	2 3/4	2.425	4	23
2	2 1/2	2.175	4	18	2 7/8	2.550	4	28
2 1/16	2 5/8	2.300	4	24	2 7/8	2.550	4	20
2 1/8	2 5/8	2.300	4	17	3	2.629	3 1/2	20
2 3/16	2 3/4	2.425	4	23	3 1/8	2.754	3 1/2	24

Table V (Continued). Standard Proportions of Upset Screw-Ends for Round and Square Bars

Diam. of round or side of square bar in	Round bars				Square bars			
	Diam. of upset screw-end in	Diam. of screw at root of thread in	Number of threads per inch	Excess of effective area of screw-end over bar %	Diam. of upset screw-end in	Diam. of screw at root of thread in	Number of threads per inch	Exce of eff tive a of scr end o ba %
2 1/4	2 7/8	2.550	4	28	3 1/8	2.754	3 1/2	[illegible]
2 5/16	2 7/8	2.550	4	22	3 1/4	2.879	3 1/2	[illegible]
2 3/8	3	2.629	3 1/2	23	3 3/8	3.004	3 1/2	[illegible]
2 7/16	3 1/8	2.754	3 1/2	28	3 3/8	3.004	3 1/2	[illegible]
2 1/2	3 1/8	2.754	3 1/2	21	3 1/2	3.100	3 1/4	[illegible]
2 9/16	3 1/4	2.879	3 1/2	26	3 5/8	3.225	3 1/4	[illegible]
2 5/8	3 1/4	2.879	3 1/2	20	3 5/8	3.225	3 1/4	[illegible]
2 11/16	3 3/8	3.004	3 1/2	25	3 3/4	3.317	3	[illegible]
2 3/4	3 3/8	3.004	3 1/2	19	3 7/8	3.442	3	[illegible]
2 13/16	3 1/2	3.100	3 1/4	22	3 7/8	3.442	3	[illegible]
2 7/8	3 5/8	3.225	3 1/4	26	4	3.567	3	[illegible]
2 15/16	3 5/8	3.225	3 1/4	21	4 1/8	3.692	3	[illegible]
3	3 3/4	3.317	3	22	4 1/8	3.692	3	[illegible]
3 1/8	3 7/8	3.442	3	21	4 3/8	3.923	2 7/8	[illegible]
3 1/4	4	3.567	3	20	4 1/2	4.028	2 3/4	[illegible]
3 3/8	4 1/8	3.692	3	20	4 5/8	4.153	2 3/4	[illegible]
3 1/2	4 1/4	3.798	2 7/8	18	...	.....	...	.
3 5/8	4 1/2	4.028	2 3/4	23	...	.....	...	.
3 3/4	4 5/8	4.153	2 3/4	23	...	.....	...	.
3 7/8	4 3/4	4.255	2 5/8	21	...	.....	...	.

REMARKS. As upsetting reduces the strength of iron, bars having the same dia at the root of the thread as that of the bar invariably break in the screw-end tested to destruction, without developing the full strength of the bar. It is the necessary to make up for this loss in strength by an excess of metal in the upset s ends over that in the bar.

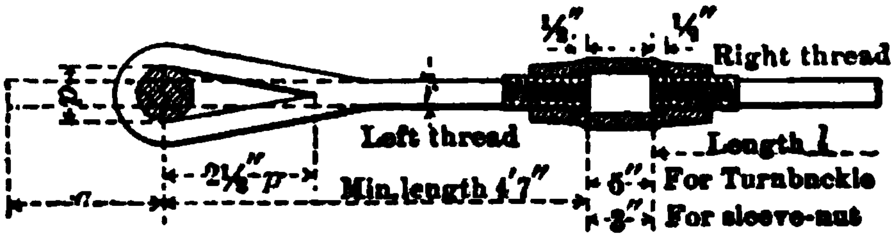
Table V is the result of numerous tests on finished bars made at the Keystone B Company's Works in Pittsburgh, Pa., and gives proportions that will cause the b break in the body rather than in the upset end.

The screw-threads in the above table are the Franklin Institute standards.

To make one upset end for a 5-in length of thread, allow 6 in in length of rod, tional.

Table VII.* Loop-Rods

AMERICAN BRIDGE COMPANY STANDARD



Pitch and shape of thread A. B. Co standard

Additional length A, in feet and inches, for one loop. $A = 4.17p + 5.89r$

Diam. of pin, p, in	Diameter or side r of rod in inches									
	¾	⅞	1	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞
1⅛	0- 9½	0-10	0-11	0-11½	....	....	....	....	....	....
1¼	0-10	0-10½	0-11½	1- 0	1- 1	....	....	....	....	....
1½	0-11	0-11½	1- 0½	1- 1	1- 2	1- 2½	....	....	....	....
1¾	1- 0	1- 0½	1- 1½	1- 2	1- 3	1- 3½	1- 4½	1- 5	1- 6	....
2	1- 1	1- 1½	1- 2½	1- 3	1- 4	1- 4½	1- 5½	1- 6	1- 7	1- 7½
2¼	1- 2	1- 3	1- 3½	1- 4½	1- 5	1- 5½	1- 6½	1- 7	1- 8	1- 8½
2½	1- 3	1- 4	1- 4½	1- 5½	1- 6	1- 7	1- 7½	1- 9	1- 9	1- 9½
2¾	1- 4	1- 5	1- 5½	1- 6½	1- 7	1- 8	1- 8½	1- 9½	1-10	1-11
3	1- 5	1- 6	1- 6½	1- 7½	1- 8	1- 9	1- 9½	1-10½	1-11	2- 0
†3¼	1- 6	1- 7	1- 7½	1- 8½	1- 9	1-10	1-10½	1-11½	2- 0	2- 1
3½	1- 7½	1- 8	1- 8½	1- 9½	1-10	1-11	1-11½	2- 0½	2- 1	2- 2
†3¾	1- 8½	1- 9	1-10	1-10½	1-11	2- 0	2- 0½	2- 1½	2- 2	2- 3
4	1- 9½	1-10	1-11	1-11½	2- 0½	2- 1	2- 2	2- 2½	2- 3	2- 4
†4¼	....	1-11	2- 0	2- 0½	2- 1½	2- 2	2- 3	2- 3½	2- 4½	2- 5
4½	....	2- 0	2- 1	2- 1½	2- 2½	2- 3	2- 4	2- 4½	2- 5½	2- 6
†4¾	....	2- 1	2- 2	2- 2½	2- 3½	2- 4	2- 5	2- 5½	2- 6½	2- 7
5	....	2- 2½	2- 3	2- 3½	2- 4½	2- 5	2- 6	2- 6½	2- 7½	2- 8
†5¼	....	....	2- 4	2- 5	2- 5½	2- 6	2- 7	2- 7½	2- 8½	2- 9
5½	....	....	2- 5	2- 6	2- 6½	2- 7½	2- 8	2- 9	2- 9½	2-10
†5¾	....	....	2- 6	2- 7	2- 7½	2- 8½	2- 9	2-10	2-10½	2-11½
6	....	....	2- 7	2- 8	2- 8½	2- 9½	2-10	2-11	2-11½	3- 0½
†6¼	....	....	....	2- 9	2- 9½	2-10½	2-11	3- 0	3- 0½	3- 1½
6½	....	....	....	2-10	2-10½	2-11½	3- 0	3- 1	3- 1½	3- 2½
†6¾	....	....	....	2-11	3- 0	3- 0½	3- 1	3- 2	3- 2½	3- 3½
7	....	....	....	3- 0	3- 1	3- 1½	3- 2½	3- 3	3- 3½	3- 4½

Pins marked † are special. Maximum shipping length of l = 35 ft.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table VIII.* Turnbuckles and Sleeve-Nuts

AMERICAN BRIDGE COMPANY STANDARD

All dimensions in inches

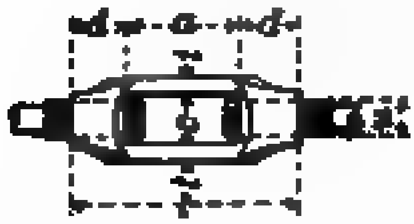
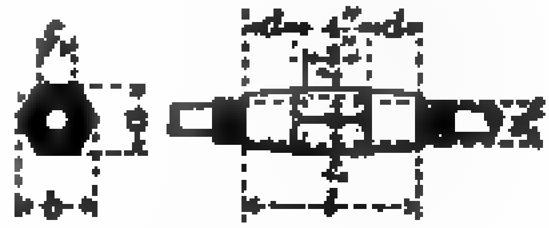
TURNBUCKLES	SLEEVE-NUTS
	
F: e=9" for turnbuckles marked† ch and shape of thread, A. B. Co standard	Pitch and shape of thread, A. B. Co standard

Table IX.* Clevises

AMERICAN BRIDGE COMPANY STANDARD

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table X. Safe Loads in Tension for Common Sizes of Angles with One ⅞-Inch Rivet-Hole for a ¾-Inch Rivet

Load in pounds for a stress of 16 000 lb per sq in

Size of angle	Load	Size of angle	Load
6×4×¾	100 500	3½×2½×⅝	45 000
⅝	85 000	9/16	41 100
½	68 900	½	37 000
		⅜	28 500
5×3½×¾	82 500	¼	19 500
⅝	69 900		
½	57 000	3×3×⅝	45 000
		½	37 000
5×3×¾	76 500	⅜	28 500
⅝	64 900	¼	19 500
½	53 000		
⅜	40 500	3×2½×½	33 000
		⅜	25 400
4×4×¾	76 500	5/16	21 600
⅜	40 500	¼	17 400
4×3½×⅝	60 000		
⅜	37 600	3×2×7/16*	25 900
		⅜*	25 600
4×3×⅝	55 000	5/16*	21 800
½	45 000	¼*	17 600
⅜	34 400		
		2½×2½×7/16	25 900
3½×3½×¾	64 500	⅜	22 400
⅝	54 900	5/16	19 200
½	45 000	¼	15 500
⅜	34 400		
		2½×2×7/16	22 400
3½×3×⅝	50 100	⅜	19 500
½	41 000	5/16	16 600
⅜	31 500	¼	13 400

*These are special angles. It is better not to use them in ordinary work because of t of delay in delivery.

he End-Connections often determine the strength of ANGLE TENSION-MBERS. Some specifications for structural work require angles subject to ct tension to be connected by both legs if the section of both legs is conred; and if connected by one leg, the section of one leg only is considered ective. Reliable tests (page 385) show this requirement to be needlessly re. For single angles connected by one leg, the Specifications for the ctural Steelwork of Buildings, Chapter XXX, allow the net area of the nected leg and one-half that of the outstanding leg to be considered effective. e Waterbury, Stresses in Structural Steel Angles, John Wiley & Sons, Inc., w York, 1917.)

Table XI. Sectional Area to be Deducted from Plates and Angles for One Round Hole

Note. Bolt-holes should be 1/16 in larger than the diameter of the bolt; rivet-h are usually 1/8 in larger than the diameter of the rivet.*

Thickness of plate	Diameter of hole in fractions of an inch and inches														
	1/4	5/16	3/8	7/16	1/2	9/16	16	3/4	1		/16	1	1 1/16	1 3/16	1 7/16	1'
3/16	0.05	0.06	0.07	0	09	0.11	13	0.14	0.		18	0.19	0.20	0.23	0.27	0
1/4	0.06	0.08	0.09	0	13	0.14	17	0.19	0		23	0.25	0.27	0.30	0.36	0
5/16	0.08	0.10	0.12	0.	16	0.18	21	0.23	0		29	0.31	0.33	0.37	0.45	0
3/8	0.09	0.12	0.14	0	19	0.21	26	0.28	0		35	0.38	0.40	0.45	0.54	0
7/16	0.11	0.14	0.16	0.	22	0.25	30	0.33	0		41	0.44	0.46	0.52	0.63	0
1/2	0.13	0.16	0.19	0	25	0.28	34	0.38	0.41	0.44	0.47	0.50	0.53	0.59	0.72	0
9/16	0.14	0.18	0.21	0	28	0.32	39	0.42	0.46	0.49	0.53	0.56	0.60	0.67	0.81	0
5/8	0.16	0.20	0.23	0	31	0.35	43	0.47	0.51	0.55	0.59	0.63	0.66	0.74	0.90	0
11/16	0.17	0.21	0.26	0.	34	0.39	47	0.52	0.56	0.60	0.64	0.69	0.73	0.82	0.99	1
3/4	0.19	0.23	0.28	0	38	0.42	52	0.56	0.61	0.66	0.70	0.75	0.80	0.89	1.08	1
13/16	0.20	0.25	0.30	0	41	0.46	56	0.61	0.66	0.71	0.76	0.81	0.86	0.97	1.17	1
7/8	0.22	0.27	0.33	0	44	0.49	60	0.66	0.71	0.77	0.82	0.88	0.93	1.04	1.26	1
15/16	0.23	0.29	0.35	0	47	0.53	64	0.70	0.76	0.82	0.88	0.94	1.00	1.11	1.35	1
1	0.25	0.31	0.38	0.	50	0.56	69	0.75	0.81	0.88	0.94	1.00	1.06	1.19	1.44	1

* See also Table I, Chapter XX and paragraph, Punching Rivet-Holes, page 414.

7. Wire

Manufacture. Iron and steel wires are made from BILLETS about 4 in squa These are rolled into long rods which are dipped in acid to remove the sc and furnish lubrication for the drawing process. This consists in pulling rods while cold through steel dies having a series of holes of gradually decreas diameters. The cold working of the metal hardens it and makes it brittle that it is necessary to anneal it at intervals during the process. The draw increases the strength of the material, so that wires of different sizes, altho made of the same material, differ greatly in ULTIMATE STRENGTH.

Finish. The common grades of iron and steel wire are furnished in sev different finishes: plain black, bright tinned, copper-coated, japanned and w single and double coats of zinc galvanizing. The last is applied by passing wire through the melted zinc which is deposited as a coating and forms on the best-known protections against corrosion.

Wire-Gauges. Table XIII gives, according to several GAUGES, the diame of the different numbers of wire that have come into use for different purp and have been brought out by different manufacturers. In ordering wire number it is best to specify which GAUGE is meant.

Strength. Table XIV gives the sizes according to the J. A. Roebling's S Company gauge, with the weight and length and the strength on an assum basis of 100 000 lb per sq in. The different kinds of wire vary so widely in u MATE STRENGTH, on account of both the difference in quality of the material the effect of the drawing, that in order to obtain the approximate strength

e, reference must be made to Table XII in connection with the foot-note to ble XIV. The following table is arranged from values which were published the Catalogue of the J. A. Roebling's Sons Company:

Table XIII. Approximate Ultimate Strength of Different Sizes of Iron and Steel Wire

Kind of wire	Ultimate strength	
	Large size lb per sq in	Small size lb per sq in
t iron	45 000	60 000
legraph and telephone (steel)	60 000	80 000
cial aviator	247 000	285 000
no wire	307 000	340 000
ugh steel wire	200 000	345 000
rd-drawn copper trolley wire	50 000	not used
rd-drawn telegraph and telephone copper	56 000	66 000

The Uses of Wire are so many and varied that a bulky treatise would equired to adequately cover the subject. The catalogues of the American el and Wire Company mention electrical wires and cables of many kinds, phone and telegraph wires, ignition-wires and cables for automobiles, motor-ts and aeroplanes, wire rope, wire tacks, wire fences, piano-wire, barbed e, flat wire and so on, through a long list. The magnitude of the wire-output the United States is seen from the fact that of the total production of 32 000 000 s of all kinds of finished rolled iron and steel for the year 1916, 3 500 000 tons e wire rods. For electrical purposes copper wire is mostly used. See ctric Work for Buildings, Part III, for information regarding wires and wire-ulations.

The Brown & Sharpe Gauge is followed in the United States as the ndard for copper wire, though there is a growing tendency to distinguish ent electrical wires by their diameters, expressed in mils. (One mil = in. A circular mil is the area of a circle 0.001 in in diameter.)

The American Steel and Wire Company's Gauge is almost universally used throughout the United States for steel wire. The Birmingham gauge, English gauge, is the only wire-gauge recognized in successive Acts of gress establishing tariffs, and for many years has been used as the basis duties assessed on imported wire. Aside from these purposes its use is not nsive. The American Steel and Wire Company's Music-Wire-Gauge, known as the Music-Wire-Gauge, upon recommendation of the United tes Bureau of Standards, has been adopted as the standard for piano-wire.

* See, also, pages 402, 403, 1469, 1473, 1509, 1510, 1512, and 1600.

Table XIII. Comparison of Standard Gauges for Wire and Sheet Metal

Number of gauge	Diameter or thickness in decimals of an inch: Birmingham or Stubs iron-wire-gauge	American or Brown & Sharpe wire-gauge	United States standard gauge for sheet and plate iron and steel	Washburn & Moen, Roebling, American Steel & Wire Co., steel-wire-gauge	Stubs steel-wire-gauge	American Screw Co. wire-gauge	Brit Imp or En leg stan wir gau
0000000	......	....	0.5	0.4900	......	......	0.5
000000	......	0.580000	0.46875	0.4615	......	......	0.4
00000	0.500	0.516500	0.4375	0.4305	......	......	0.4
0000	0.454	0.460000	0.40625	0.3938	......	......	0.4
000	0.425	0.409642	0.375	0.3625	......	0.0315	0.3
00	0.380	0.364796	0.34375	0.3310	......	0.0447	0.3
0	0.340	0.324861	0.3125	0.3065	......	0.0578	0.3
1	0.300	0.289297	0.28125	0.2830	0.227	0.0710	0.3
2	0.284	0.257627	0.265625	0.2625	0.219	0.0842	0.2
3	0.259	0.229423	0.25	0.2437	0.212	0.0973	0.2
4	0.238	0.204307	0.234375	0.2253	0.207	0.1105	0.2
5	0.220	0.181940	0.21875	0.2070	0.204	0.1236	0.
6	0.203	0.162023	0.203125	0.1920	0.201	0.1368	0.1
7	0.180	0.144285	0.1875	0.1770	0.199	0.1500	0.1
8	0.165	0.128490	0.171875	0.1620	0.197	0.1631	0.1
9	0.148	0.114423	0.15625	0.1483	0.194	0.1763	0.1
10	0.134	0.101897	0.140625	0.1350	0.191	0.1894	0.
11	0.120	0.090742	0.125	0.1205	0.188	0.2026	0.1
12	0.109	0.080808	0.109375	0.1055	0.185	0.2158	0.
13	0.095	0.071962	0.09375	0.0915	0.182	0.2289	0.
14	0.083	0.064084	0.078125	0.0800	0.180	0.2421	0.
15	0.072	0.057068	0.0703125	0.0720	0.178	0.2552	0.
16	0.065	0.050821	0.0625	0.0625	0.175	0.2684	0.
17	0.058	0.045257	0.05625	0.0540	0.172	0.2816	0.
18	0.049	0.040303	0.05	0.0475	0.168	0.2947	0.
19	0.042	0.035890	0.04375	0.0410	0.164	0.3079	0.
20	0.035	0.031961	0.0375	0.0348	0.161	0.3210	0.
21	0.032	0.028462	0.034375	0.0317	0.157	0.3342	0.
22	0.028	0.025346	0.03125	0.0286	0.155	0.3474	0.
23	0.025	0.022572	0.028125	0.0258	0.153	0.3605	0.
24	0.022	0.020101	0.025	0.0230	0.151	0.3737	0.
25	0.020	0.017900	0.021875	0.0204	0.148	0.3868	0.
26	0.018	0.015941	0.01875	0.0181	0.146	0.4000	0.
27	0.016	0.014195	0.0171875	0.0173	0.143	0.4132	0.
28	0.014	0.012641	0.015625	0.0162	0.139	0.4263	0.
29	0.013	0.011257	0.0140625	0.0150	0.134	0.4395	0.
30	0.012	0.010025	0.0125	0.0140	0.127	0.4526	0.
31	0.010	0.008928	0.0109375	0.0132	0.120	0.4658	0.
32	0.009	0.007950	0.01015625	0.0128	0.115	0.4790	0.
33	0.008	0.007080	0.009375	0.0118	0.112	0.4921	0.
34	0.007	0.006305	0.00859375	0.0104	0.110	0.5053	0.
35	0.005	0.005615	0.0078125	0.0095	0.108	0.5184	0.
36	0.004	0.005000	0.00703125	0.0090	0.106	0.5316	0.
37	......	0.004453	0.006640625	0.0085	0.103	0.5448	0.
38	......	0.003965	0.00625	0.0080	0.101	0.5579	0.
39	......	0.003531	....	0.0075	0.099	0.5711	0.
40	......	0.003144	....	0.0070	0.097	0.5842	0.

The United States Standard Gauge was legalized by Act of Congress, March 3, s a standard gauge for sheet and plate iron and steel, and is used by the Custom)epartment and by sheet-plate and tin-plate manufacturers.

* See also, pages 401, 403, 1469, 1473, 1509, 1510, 1512, and 1600.

Table XIV. Weight, Length and Strength of Steel Wire *

Gauge of J. A. Roebling's Sons Company †

Number of gauge	Diameter in	Area sq in	Breaking-load in pounds at rate of 100 000 lb per sq in	Weight in pounds		Number of feet in 2 000 pounds
				Per 1 000 ft	Per mile	
000000	0.460	0.166191	16 619	558.4	2 948	3 582
00000	0.430	0.145221	14 522	487.9	2 576	4 099
0000	0.394	0.121304	12 130	407.6	2 152	4 907
000	0.362	0.102922	10 292	345.8	1 826	5 783
00	0.331	0.086049	8 605	289.1	1 527	6 917
0	0.307	0.074023	7 402	248.7	1 313	8 041
1	0.283	0.062902	6 290	211.4	1 116	9 463
2	0.263	0.054325	5 433	182.5	964	10 957
3	0.244	0.046760	4 676	157.1	830	12 730
4	0.225	0.039761	3 976	133.6	705	14 970
5	0.207	0.033654	3 365	113.1	597	17 687
6	0.192	0.028953	2 895	97.3	514	20 559
7	0.177	0.024606	2 461	82.7	437	24 191
8	0.162	0.020612	2 061	69.3	366	28 878
9	0.148	0.017203	1 720	57.8	305	34 600
10	0.135	0.014314	1 431	48.1	254	41 584
11	0.120	0.011310	1 131	38.0	201	52 631
12	0.105	0.008659	866	29.1	154	68 752
13	0.092	0.006648	665	22.3	118	89 525
14	0.080	0.005027	503	16.9	89.2	118 413
15	0.072	0.004071	407	13.7	72.2	146 198
16	0.063	0.003117	312	10.5	55.3	191 022
17	0.054	0.002290	229	7.70	40.6	259 909
18	0.047	0.001735	174	5.83	30.8	343 112
19	0.041	0.001320	132	4.44	23.4	450 856
20	0.035	0.000962	96	3.23	17.1	618 620

...is table was calculated on a basis of 483.84 lb per cu ft for steel wire. Iron wire is ...le lighter.

...e breaking strengths were calculated for 100 000 lb per sq in throughout, simply ...venience, so that the breaking strengths per square inch of wires of any strength ...be quickly determined by multiplying the values given in the table by the ratio ...en the strength per square inch and 100 000. Thus, a No. 15 wire, with a strength ...square inch of 150 000 pounds, has a breaking strength of

$$407 \times \frac{150\,000}{100\,000} = 610.5 \text{ lb.}$$

...must not be inferred from this table that steel wire invariably has a strength of ...00 lb per sq in. As a matter of fact its strength ranges from 45 000 lb per sq in for ...annealed wire to over 400 000 lb per sq in for hard wire.

* See, also, pages 401, 402, 1469, 1473, 1509, 1510, 1512, and 1600

† Also American Steel & Wire Company, etc.

8. Wire Rope

Kinds of Wire Rope. There are several kinds of WIRE ROPE in com use. In each there are three or more qualities depending on the kind of used and the kind of core about which the strands are laid. The Trenton Company lists the following:

(1) **Haulage or Transmission-Rope,** composed of six strands of seven each, laid about a hemp core. It is used for haulage, transmission of powe places where surface-wear is of chief consideration and where sheaves of suffi diameter may be used.

(2) **Hoisting-Rope,** composed of six strands of nineteen wires each. used for elevator service, shafts and derricks, and in places where it is not ject to abrasion and where flexibility is of chief consideration.

(3) **Seale Rope,** composed of six strands of nineteen wires each, the coils of the strands being of finer wire. It is intermediate in flexibility bet the first and second kinds of rope.

(4) **Non-Spinning Hoisting-Rope,** having eighteen strands of seven wires Twelve of the strands are laid in reverse direction to the inner six, ma it well adapted for hoisting in free suspension without untwisting and tur the load.

(5) **Extra-Flexible Hoisting-Rope,** having eight strands of nineteen wires

(6) **Special Flexible Hoisting-Rope,** having six strands of thirty-seven each.

(7) **Hawser-Rope and Flexible Running-Rope,** having six strands of t galvanized wires each, laid about a hemp core.

(8) **Tiller-Rope,** composed of six small seven-strand ropes laid about a core. It is the most flexible of wire ropes and is used to operate tillers an hand-ropes in elevators.

The Lay of Wire Rope is the twist of the wires in the strands relativ the strands in the rope. In the ORDINARY LAY the twist of the strands reverse of that of the wires, while in the LANG LAY the strands are laid i same direction as the twist of the wires. This latter gives a greater distrib of the wearing-surface and a somewhat greater flexibility; but it has th advantage of a tendency to untwist and for this reason should not be use hoisting weights in free suspension. Wire rope is also made up in FLAT O BON FORM. For large sizes it is more flexible than standard rope and m run over smaller drums.

Materials for Rope. Nearly all of the above kinds of rope are made the following materials:

(1) **Best Grade of Wrought Iron.** This is used in high-speed PASSE ELEVATOR SERVICE as it seems to suffer less from the effects of the stresse to the starting and stopping of the cars.

(2) **Cast-Steel Wire,** with an ultimate strength of from 160 000 to 210 c per sq in, according to the size used.

(3) **Extra-Strong Cast-Steel Wire,** with an ultimate strength of from 19 to 230 000 lb per sq in.

(4) **Plow-Steel Wire** with an ultimate strength of from 200 000 to 230 per sq in.

Ordinary GALVANIZED-WIRE ROPE should not be used for other than sta rope. A short service running through sheaves will break the coating and

Table XV. Strength of Wire Rope

Arranged from the 1912 list of John A. Roebling's Sons Company

Trade number	Diameter inches	Weight per foot, hemp core	Approximate breaking-load in pounds		Minimum diameter of drum or sheave in feet	
			Iron	Cast steel	Iron	Cast steel
HOISTING-ROPE — Six strands of nineteen wires each, about a hemp core						
1	2¼	8.00	144 000	266 000	14	9
2	2	6.30	110 000	212 000	12	8
2½	1⅞	5.55	100 000	192 000	12	8
3	1¾	4.85	88 000	170 000	11	7
4	1⅝	4.15	76 000	144 000	10	6.5
5	1½	3.55	66 000	128 000	9	6
5½	1⅜	3	56 000	112 000	8.5	5.5
6	1¼	2.45	45 600	94 000	7.5	5
7	1⅛	2	37 200	76 000	7	4.5
8	1	1.58	29 000	60 000	6	4
9	⅞	1.20	23 600	46 000	5.5	3.5
10	¾	0.89	17 000	35 000	4.5	3
10¼	⅝	0.62	12 000	25 000	4	2.5
10½	9/16	0.50	9 400	20 000	3.5	2.25
10¾	½	0.39	7 800	16 800	3	2
10a	7/16	0.30	5 800	13 000	2.75	1.75
10b	⅜	0.22	4 800	9 600	2.25	1.5
STANDING ROPE — Six strands of seven wires each						
11	1½	3.55	64 000	126 000	16	11
12	1⅜	3	56 000	106 000	15	10
13	1¼	2.45	46 000	92 000	13	9
14	1⅛	2	38 000	74 000	12	8
15	1	1 58	30 000	62 000	10.5	7
16	⅞	1.20	24 000	48 000	9	6
17	¾	0.89	17 600	37 200	7.5	5
18	11/16	0.75	14 600	30 800	7.25	4.75
19	⅝	0 62	12 000	26 000	7	4.50
20	9/16	0.50	9 600	20 000	6	4
21	½	0.39	7 400	15 400	5.5	3.5
22	7/16	0.30	5 200	11 000	4.5	3
23	⅜	0.22	4 400	9 200	4	2.75
24	5/16	0.15	3 400	7 000	3.5	2.25
25	9/32	0.125	2 400	5 000	3	1.75

e working load is to be taken at one-fifth the breaking-load. This is assumed i
lating the diameter of the sheaves.

rapid corrosion of the rope. Because of the many kinds and qualities of r
it is well to consult the manufacturers as to which kind will best suit the co
tions for any particular service. The John A. Roebling's Sons Company, T
ton, N. J., the Trenton Iron Company, Trenton, N. J., and A. Leschen & S
Rope Company, St. Louis, Mo., are among the largest manufacturers of
lines of ropes.

Coils. Wire rope should not be coiled like hemp rope, and in order to a
kinking, should be taken from the reels without twisting. If it is not shippe
a reel, to avoid injury it must be rolled over the ground like a wheel.

Lubrication. It is very important that running ropes be properly lu
cated, since, if proper care is not taken, the wear on the interior parts, betw
the wires, may be almost as great as the outside abrasion. The oil should p
trate to the core of the rope and yet not drip off a few days after applica
Information as to the care of rope may be obtained of the Wire Rope Lub
ting Company, Newark, N. J.

Sheaves. The size of sheaves recommended in the tables are calculated
a working-load of one-fifth the given breaking-load. If smaller sheaves are
the life of the rope will be greatly shortened, because of the excessive ben
of the outer wires.

Table XVI. Galvanized, Steel-Wire Strands

For guys, signal-cords, trolley-line span wire, etc. Taken from the American Steel & Iron Company's list

Diameter in inches	List price per 100 feet *	Weight per 100 feet	Approximate breaking-load in pounds
5/8	$7.25	80	14 000
9/16	5.75	65	11 000
1/2	4.50	52	8 500
7/16	3.75	41	6 500
3/8	2.75	30	5 000
5/16	2.25	22	3 800
1/4	1.75	13	2 300
7/32	1.50	9½	1 800
3/16	1.25	7½	1 400
5/32	1.15	5½	900
1/8	1.00	3¼	600
3/32	.80	2	400

9. Cotton, Hemp and Manila Rope

Rope is made of cotton, hemp, and Manila fiber. Cotton is used for
sizes, only, and for such purposes as sash-cord, etc.

Manufacture. In the manufacture of rope the fiber is first spun into
From twenty to eighty threads are twisted together into STRANDS an
strands, three or four, are laid together, opposite in direction to the TW
the strands, but in the same direction as the THREADS. This causes the
to be twisted as the rope untwists and produces a balancing of forces tha
to keep the rope in shape.

Cables and Hawsers are made up of strands of rope.

* These pre-war prices must be increased as per current price-lists.

pe used for Hoisting wears rapidly from the action of the pulleys and from the bending which causes a slight internal motion between the fiber a chafing and grinding away of the interior.

tevedore-Rope, of the C. W. Hunt Company, is filled with a tallow and ago lubricant which decreases the internal friction, lubricates the outside e rope and thus greatly prolongs its life.

rength. The values of the strength of new rope, given in Table XVII, are n from the Specifications of the United States Navy Department, issued ne, 1910, at the Boston Navy-Yard. Manufacturers generally adopt these and weights and claim a strength equal to or a little greater than the values a. The UNIT STRENGTH for the different sizes varies, being about 14 000 lb q in for the smaller and about 10 000 for the largest size. The approx e formula, offered by C. W. Hunt, of 720 times the square of the circum ce in inches, is equivalent to about 9 000 lb per sq in. American hemp d, has about 5% greater strength than Manila rope of the same size. The specifications give for two-strand American-hemp rope, 85% of the strength e three-strand rope of the same material.

Table XVII. Strength and Weight of Rope

Specifications of the United States Navy, June, 1910

Circum-ferences in	Diameters in	Manila hemp, plain-laid: Weights lb per ft	Manila hemp, plain-laid: Breaking-loads lb	American hemp, tarred, plain-laid, three strands: Weights lb per ft	American hemp, tarred, plain-laid, three strands: Breaking-loads lb
3/4	0.24	0.02	700	0.051	750
1	0.32	0.033	1 000	0.06	1 060
1 1/4	0.40	0.05	1 800	0.067	1 670
1 1/2	0.48	0.083	2 500	0.083	2 340
1 3/4	0.56	0.10	3 000	0.105	3 325
2	0.64	0.14	4 000	0.16	3 955
2 1/4	0.72	0.17	5 000	0.21	4 720
2 1/2	0.80	0.21	5 500	0.26	5 770
2 3/4	0.87	0.26	6 600	0.32	7 000
3	0.95	0.305	7 800	0.37	8 400
3 1/4	1.03	0.36	9 200	0.44	9 800
3 1/2	1.16	0.42	10 500	0.51	11 200
3 3/4	1.19	0.47	12 200	0.59	13 000
4	1.27	0.54	13 700	0.67	14 550
4 1/2	1.43	0.67	17 400	....	......
5	1.59	0.83	21 800	....	......
5 1/2	1.75	1.00	27 700	....	......
6	1.90	1.21	31 000	....	......
7	2.22	1.63	36 200	....	......
8	2.54	2.17	47 300	....	......
9	2.87	2.70	60 000	....	......
10	3.14	3.33	74 200	....	......

ila-hemp rope is made in three strands and in sizes up to 3 in in circumference ands are used for sizes larger than 3 in in circumference.

Working Load. The WORKING LOAD for slow-speed derrick and hoi service is usually taken at one-seventh the BREAKING-LOAD. This makes allowance for the loss of strength at splices and connections. The deterio of rope exposed to the weather is very rapid. For Manila rope from 1 to in diameter, running over sheaves of the diameters given, C. W. Hunt in T Am. Soc. M. E., Vol. XXIII, gives a table embodying approximately th lowing results of experience:

Table XVIII. Working Loads for Manila Rope

Working load = C × breaking-load of new rope

D = minimum diameter of sheave in inches

Speed	Feet per minute	Kind of work	Value of C	D for rop diamete 1 in	1
Slow......	50 to 100	Derrick, crane, quarry, etc.	0.014	8	
Medium...	150 to 300	Wharf, cargo, etc.	0.056	12	
Rapid.....	400 to 800	....	0.028	40	

The wear in such service is very rapid, a 1½-in rope wearing out in liftin 7 000 to 10 000 tons of coal. On the other hand, a 1½-in transmissio running at 5 000 ft per min and carrying 1 000 horse-power over sheave and 17 ft in diameter, lasts for years, the difference being due to the s stress and larger sheaves.

10. Chains

Manufacture. Large chains are made by hand-welding from best wr iron bar, and small chains up to ½ in are best made of mild open-hearth electric-welded. The bending and welding reduce the strength so th chain is not twice but only from 1.55 to 1.70 times as strong as the origin from which it was made. STUD CHAIN having a bar welded across eac to stiffen it and prevent fouling in handling, is not as strong as OPE CHAIN, but has a higher elastic limit and working strength. G. A. Goode in a Bulletin from the Illinois Engineering Experiment-Station, finds the mum stresses at the elastic limit of the material to be as follows: If P load, d the diameter of the bar, and S the stress, the formulas are:

$$P = 0.5\, d^2S \text{ for stud-link, and}$$
$$P = 0.4\, d^2S \text{ for open link.}$$

Proof-Tests. A proof-test is applied to chains by the manufacturers load applied is one-half the average BREAKING-LOAD. It serves to dete welds and gives a chain a slight permanent set, so that for working loads after there will be little stretching of the chain.

Care of Chains. Chains in constant use require lubrication and fr annealing. They harden in service and are liable to unexpected failure annealed. It is recommended that hoisting chains and sling chains be an at least once a year.

Table XIX. Sizes, Weights, Proof-Tests and Average Breaking-Loads for Chains

Bradlee and Company, Philadelphia

Size of chains in	Approximate weight per foot	D.B.G. special crane Proof-test lb	D.B.G. special crane Average breaking-load lb	Crane Proof-test lb	Crane Average breaking-load lb
1/4	3/4	1 932	3 864	1 680	3 360
3/8	1½	4 186	8 372	3 640	7 280
1/2	2.5	7 728	15 456	6 720	13 440
5/8	4.1	11 914	23 828	10 360	20 720
3/4	6.2	17 388	34 776	15 120	30 240
7/8	8.4	22 484	44 968	20 440	40 880
1	10.5	29 568	59 136	26 880	53 760
1 1/8	13.6	37 576	75 152	34 160	68 320
1 1/4	16	46 200	92 400	42 000	84 000
1 3/8	19.2	55 748	111 496	50 680	101 360
1 1/2	23	66 528	133 056	60 480	120 960
1 5/8	28	74 382	148 764		
1 3/4	31	82 320	164 640		
1 7/8	35	94 360	188 720		
2	40	107 520	215 040		
2 1/8	46.5	121 240	242 480		

The specifications of the United States Navy Department require the same proof-test as given above for crane-chain and a breaking-strength 10% greater than that given for special crane-chain.

Table XX. Proof-Tests and Average Breaking-Loads for Studded Chain-Cables

Specifications of the United States Navy Department

Size of cable in	Proof-test lb	Average breaking-load lb	Size of cable in	Proof-test lb	Average breaking-load lb
1	34 607	67 526	1 15/16	130 202	225 687
1 1/8	43 812	82 686	2	138 739	239 732
1 1/4	54 194	100 630	2 1/16	147 544	254 223
1 5/16	59 784	109 771	2 1/8	156 622	269 160
1 3/8	65 574	119 355	2 1/4	175 591	300 373
1 7/16	71 672	129 385	2 1/2	216 779	368 153
1 1/2	78 041	139 861	2 5/8	238 995	404 719
1 9/16	84 678	150 783	2 3/4	262 302	443 069
1 5/8	91 588	162 152	2 7/8	286 692	483 203
1 3/4	106 222	186 228	3	312 165	525 121
1 7/8	121 937	212 188	3 1/8	339 102	567 823

Factors of Safety. For dead loads the FACTOR OF SAFETY may be a as four provided the breaking of the chain would not imperil life. This factor generally quoted in catalogues, but is too low for most purposes MAXIMUM FIBER-STRESS is then well above the ELASTIC LIMIT of chain Where loads are to be raised repeatedly with machinery which can be op without jerks or sudden change of speed, the use of a factor of six is practice. If a chain must be used where shocks occur, the INSTANTA LOAD should be calculated, and a high factor of safety employed.

Grades of Chain. Chains up to 1 ¼ in are usually made in three g called PROOF, BB, and BBB. The proof is the cheapest grade, and is m longer links than the others. This is not ordinarily proof-tested. BB next grade, somewhat shorter linked, and is proof-tested. BBB is shorter link and more carefully made.

Crane Chain is finished in such a way as to be without twist when h with one end free, so that hooks and fittings are always facing their direction.

Dredge Chain is straightened as is Crane Chain, and made with u links to run over a wheel.

Steel Loading Chain is made mostly in small sizes for use where the compared to the strength is to be a maximum. It is the highes hand-made chain.

Block Chain is fitted to the pocket-wheel in which it is to run. In sizes it is usually electric-welded.

Electric-Welded Chain is made in small sizes and is rapidly replaci hand-made below ½ in. It is stronger and more uniform.

Sizes of Chain. Chain is ordinarily made of wire or rod, 1/32 in large the NOMINAL DIAMETER, by which it is called. If chain is desired made of the size by which it is called, it must be specified as EXACT SIZE. Steel I Chain, Block Chain, and frequently Dredge Chain, are made EXACT. Link Anchor Chain is made of wire, 3/64 in above its NOMINAL DIAMETER

CHAPTER XII

RESISTANCE TO SHEAR. RIVETED JOINTS. PINS AND BOLTS

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1. Shear

Shear is the internal stress in a body which resists the tendency of two adjacent parts to slide on each other, due to the action of two equal and parallel external forces, called SHEARING-FORCES, acting on opposite sides of the plane of shear.

If the piece *abcd* of Fig. 1 represents a short simple beam of brittle material on which a sufficient load is applied, it will fail in VERTICAL SHEAR at *f* and *g*, as shown, by a sliding on the sections of the beam at these points, because the upward force of the reaction at *S* and the downward force of the load adjacent to *S*, against which it acts across the section at *S*, is greater than the total SHEARING RESISTANCE of the section.

Fig. 1. Shearing-failure of Beam

Shear is present over the entire length of the beam, and at any section is equal to the reaction at *S* minus the weight of the load between the reaction and the section in question. In general, the VERTICAL SHEAR at any section of a beam subjected to vertical loads is equal to the algebraic sum of all the vertical forces acting on the beam on either side of the section.

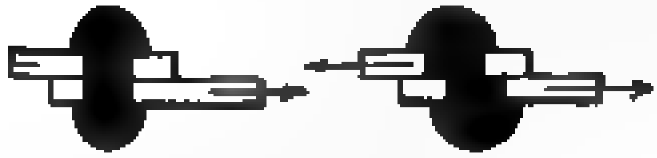


Fig. 2. Example of Single Shear

Single and Double Shear. A rivet connecting two bars under tension (Fig. 2) is subjected to a SHEARING-STRESS. If one section of the rivet transmits the force the rivet is said to be in SINGLE SHEAR; if two sections, it is in DOUBLE SHEAR.

Distribution of Shear. Shear is considered to be UNIFORMLY DISTRIBUTED over the section except in cases of torsion and of complex stresses.

For the ordinary cases of shear in rivets, etc., if

S_s = the allowable unit stress in shear,
A = the area under stress,
P = the safe shearing-load;

$$P = AS_s \qquad (1)$$

The Ultimate Strength in Shear has been determined for building materials by testing suitably prepared specimens and dividing the maximum load ob-

served by the area under stress. For material like wood, in which there planes of weakness, tests must be made which take these into account. direction of the force with respect to these planes must be considered in ch ing the SAFE WORKING STRESS from the tables.

Safe Working Stresses in Shear. Table I gives SAFE WORKING STRES in SHEAR for those building materials usually subjected to such stresses.

Table I. Safe Working Stress in Shear for Building Materials*

Material	Safe stress in lb per sq in	
Cast iron (New York)	3 000	
Wrought iron	7 500	
Steel, bolts, rivets	10 000 (average)	
	With the grain	Across th grain
White oak	200	1 000
White pine	100	500
Long-leaf yellow pine	150	1 250
Short-leaf yellow pine	130	1 000
Douglas fir	100	900
Hemlock	100	600
Spruce	100	750

* Note. For woods, these values may be increased up to 30% for selected, pert protected, commercially dry timber, not subject to impact, that is, for ideal condi (See, also, pages 637 and 647.)

Shear in Wooden Tie-Beams. There are a few cases in architectural struction in which the weakness of wood in shear must be provided for. one most frequently arising i framing of the end of the beams in wooden trusses.

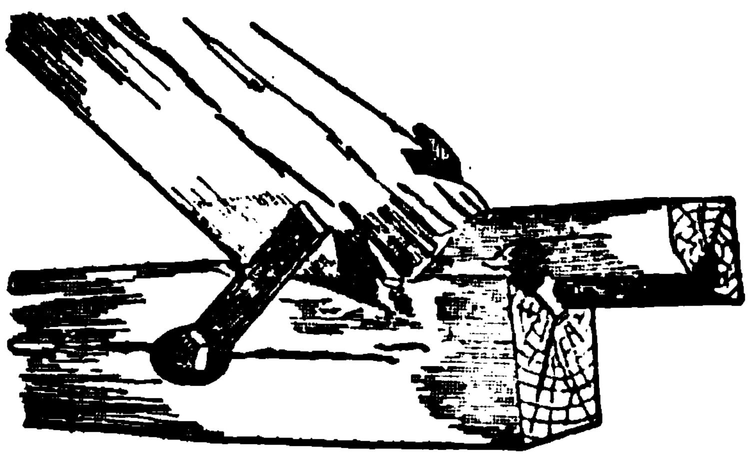


Fig. 3. Shearing-failure in Wood

Fig. 3 was made fro photograph of a SHEAI FAILURE of a tie-beam from thrust of the rafter.

Horizontal Shear Wooden Beams. Failure that shown in Fig. 1 r occurs in wood; but rectan wooden beams, the lengt which is less than about twenty times the depth, are liable to fa HORIZONTAL SHEAR along the middle, under about the same loads that the allowable working stresses in bending.

Shear at the End of a Tie-Beam. In the case of the truss-joint (Fi the thrust S of the rafter tends to shear off the part $ABCD$ along the pla which CD is the trace. This area under stress must offer a SHEARING RE ANCE equal to the horizontal component H of the thrust S. The width beam b, being fixed, formula (1) gives

$$H = (CD \times b)S_s \quad \text{or} \quad CD = H/bS_s$$

The shear being in the same direction as the grain of the wood, the lower in Table I must be used.

Example 1 (Fig. 4). The horizontal component of the thrust of a rafter is 20 000 lb. The long-leaf yellow pine tie-beam is 10 in wide. How far should the beam extend beyond the point D?

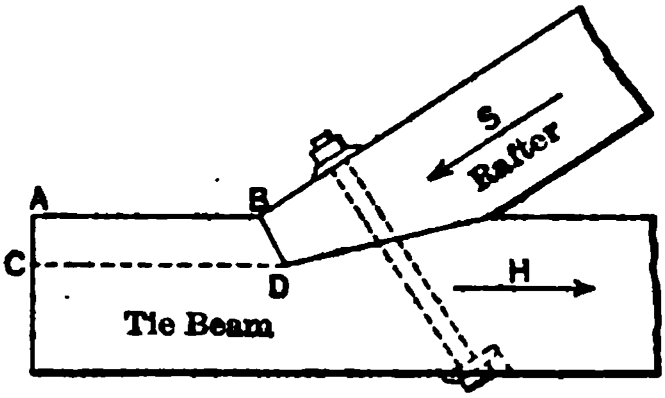


Fig. 4. Truss-joint

Solution. In this case $H = 20\,000$ lb. From Table I, $S_s = 150$ lb per sq in. Then $CD = \dfrac{20\,000}{10 \times 150} = 13.3$ in and should be made at least $13\frac{1}{2}$ in.

As actually constructed a large part of the thrust is generally taken by a bolt or strap at the foot of the rafter to hold it in place. As the bolt and shoulder seldom act together, either the length CD on the tie-beam should be made long enough to resist the entire thrust, or the bolt or strap designed to do so without relying on the shearing resistance in the plane of CD. The design of such joints is more fully considered under Subdivision 4, pages [4]39 to 439 of this chapter.

2. Riveted Joints

Use of Rivets. Rivets almost exclusively are used in connecting the plates and shapes which make up the members of framed steel construction.

Rivet-Definitions. A rivet is a piece of cylindrical rod with a HEAD forged on one end and usually with a slight taper at the other end of the SHANK. The GRIP (Table IV) of the rivet is the length betweeen the under sides of the heads after driving, or the thickness of the parts joined. The LENGTH (Table IV) of the rivet is equal to the grip plus enough of the stock to form a head, and is measured from the end of the shank to the under surface of the head. The DIAMETER OF THE SHANK of a rivet is made equal to its NOMINAL DIAMETER, but rivets are driven into holes $\frac{1}{16}$ in larger in diameter and upset by the driving so as to completely fill the holes. The shearing values and bearing values are based upon the NOMINAL AREA and not upon the area of the hole.

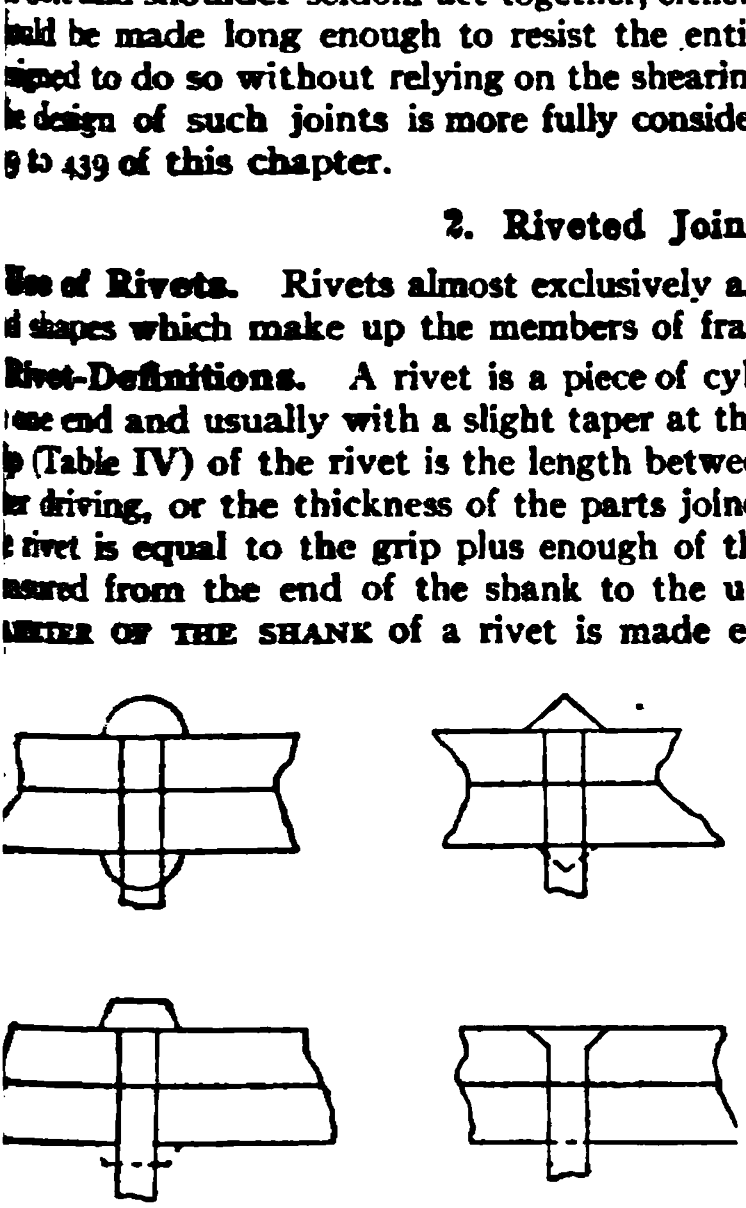


Fig. 5. Forms of Rivet-heads

Riveting consists in heating the rivet to a welding-heat, passing it through holes in the parts to be joined and forging another head out of the projecting shank. This may be done by hand-hammering; but shops use compressed-air-operated hand-hammers or large riveting-machines which form the head and cause the shank to completely fill the hole by heavy pressure on a die.

Material of Rivets. Rivets are made of soft steel and of wrought iron. Soft-steel is generally used. The head may have any of the forms shown in Fig. 5, although the first, called the BUTTON-HEAD, is the standard for structural work. The fourth or COUNTERSUNK HEAD is used where it is necessary to have a flat surface, as over a bearing-plate.

The Sizes of Rivet-Heads differ slightly at different mills. The Standa of the American Bridge Company give for the DIAMETER OF THE HEAD, one a one-half times the diameter of the shank plus ⅛ in, and for the HEIGHT of head, 0.425 times the diameter of the head. Countersunk heads have a SLO of 30° and a DEPTH equal to one-half the diameter of the shank.

The Pitch of Rivets. By this is meant the center-to-center dista between them in a line of riveting. The distance between lines of rivets, from the back of an angle or channel to a rivet-line is called the GAUGE-D TANCE. By STAGGERED PITCH is meant the arrangement of rivets midv between others on successive rivet-lines in order to decrease the section than when they are arranged in rectangular rows, and at the same time to p a greater number of rivets in a definite area. The PITCH should not be m less than three diameters of the rivet and the DISTANCE FROM THE EDGE of plate not less than one and one-half diameters, although it may be neces to make the distance less when small angles are used. The pitch of coun sunk rivets must be greater than that of button-head rivets because of greater amount of material removed.

Punching Rivet-Holes. Rivet-holes are made with power-punches. SPACING is marked on the different parts to be fastened together by mea wooden templates with holes drilled to locate the position of the rivets. W the different parts are assembled, the holes are laid out by the same TEMPL REGISTER, so that the rivets may be inserted without difficulty. PUNC makes a ragged hole. The flow of the metal under the great pressure har it and causes a loss in strength of from 11 to 33% as reported by W. C. U for soft steel. The injury may be removed by ANNEALING or by REAMING the injured part of the metal. Enlarging a ⅝-in. hole by reaming to 1 has been found to remove all the injurious effects of punching. One me practiced in the best work is to punch the holes 1/16 in less in diameter than diameter of the rivets, and to ream them to a diameter 1/16 in greater, afte parts are assembled and bolted together. This removes the greater pa the injury from punching and corrects the alinement of the holes. Table XI, page 400, and Table I, page 702.)

Drift-Pins. When the alinement of a hole is such as to prevent the inse of the rivet, it is the practice in some shops to drive in a tapered DRIFT-PI distort the holes in some of the plates sufficiently to set the rivet. This c LOCAL STRESSES and injury to the plates and should not be permitted.

Shop-Riveting is done with powerful air or hydraulic riveting-mac which may exert a pressure of from 30 to 50 tons, sufficient to upset a p head on the projecting end of the shank and to completely fill the hole though the alinement is imperfect. Contraction on cooling causes great sure between the parts, so that it is probable that in good work the rivet is little or no shearing-stress, the force being transmitted through the fric resistance of the plates.

Clearance. It is important that the designer place the rivets so they be inserted from one side and pounded on the other for HAND-WORK, or s the machine may reach them for MACHINE-RIVETED WORK. For exampl minimum distance from the inside face of the leg of one angle to a line of in the other leg must not be less than 1⅛ in for ⅞-in rivets, 1 in for ¾-in etc. In general, a distance ⅝ in greater than the diameter of the head be allowed for CLEARANCE.

Inspection. The common imperfections in riveting are LOOSE RIVET ECCENTRIC HEADS. Loose rivets may be detected by holding the hand a

e side of the rivet-head and tapping the other side with a light hammer. loose, a slight slip may be felt. The loose rivets should be marked to be cut t and replaced. The inspector should also carefully check open holes left r field-connections, and see that flattened and countersunk rivets are as called r, because such work may be done at less expense in the shop than in the ld, where it may cause delay.

The Failure of Riveted Joints may occur

(1) In TENSION, by the tearing of the plate through the line of rivets (Fig. 8).

(2) In SHEAR, by the cutting of the rivets (Fig. 7).

(3) In BEARING, by the crushing of the plate in front of the rivets, the splitting of the plate, or, in some cases, by the shearing out of the sections in front the rivets. In a careful design of a joint the strength against failure by each these methods must be investigated (Fig. 6 and Fig. 9).

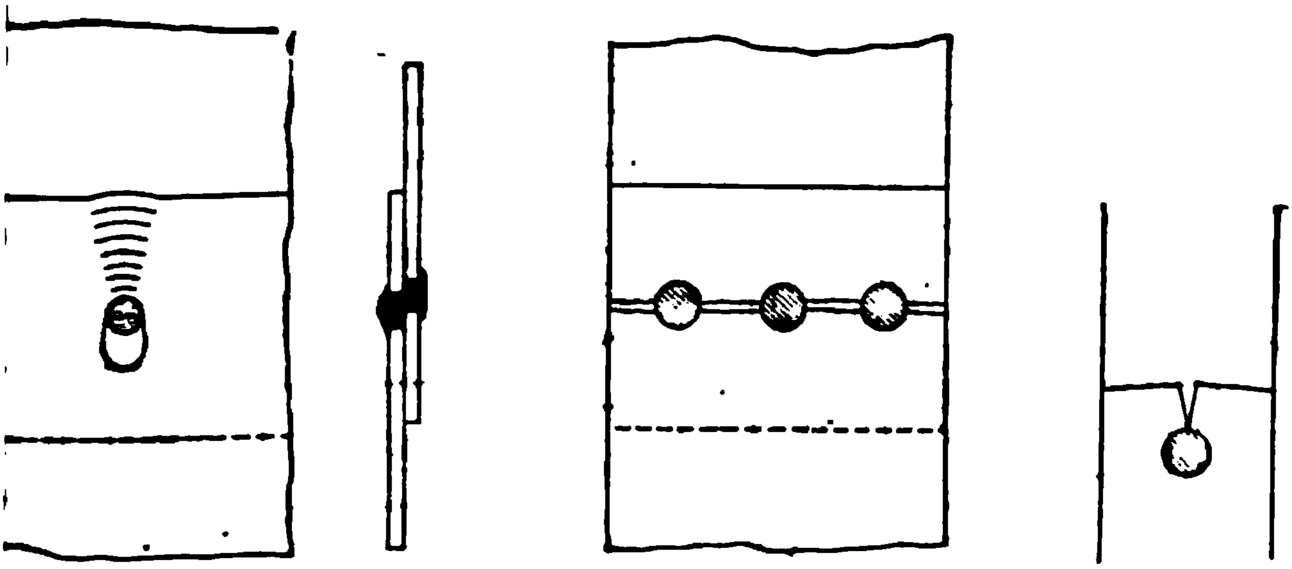


Fig. 6 Fig. 7 Fig. 8 Fig. 9

Figs. 6 to 9. Methods of Failure in Riveted Joints

The Steps in the Design of any type of riveted joint are, (1) the selection of the size of the rivet to be used, (2) the determination of its shearing bearing strength and the use of the smaller value of the two to divide into total load to be transmitted and thus determine the number of rivets, the arrangement of the rivets in the plate and the investigation of its strength tension at the dangerous section.

The Size of Rivets is determined in part by SHOP-PRACTICE. Holes cannot be punched in plates which are thicker than the diameter of the punch. following table gives the size of rivets used with plates of different thickness. e specifications for structural work require all rivets to be ¾ in, except re thick plates require larger ones.

Thickness of plates	Size of rivets
¼ to 7/16 in	⅝ in
½ to ⅝ in	¾ in
11/16 to 13/16 in	⅞ in
⅞ to 1 in	1 in

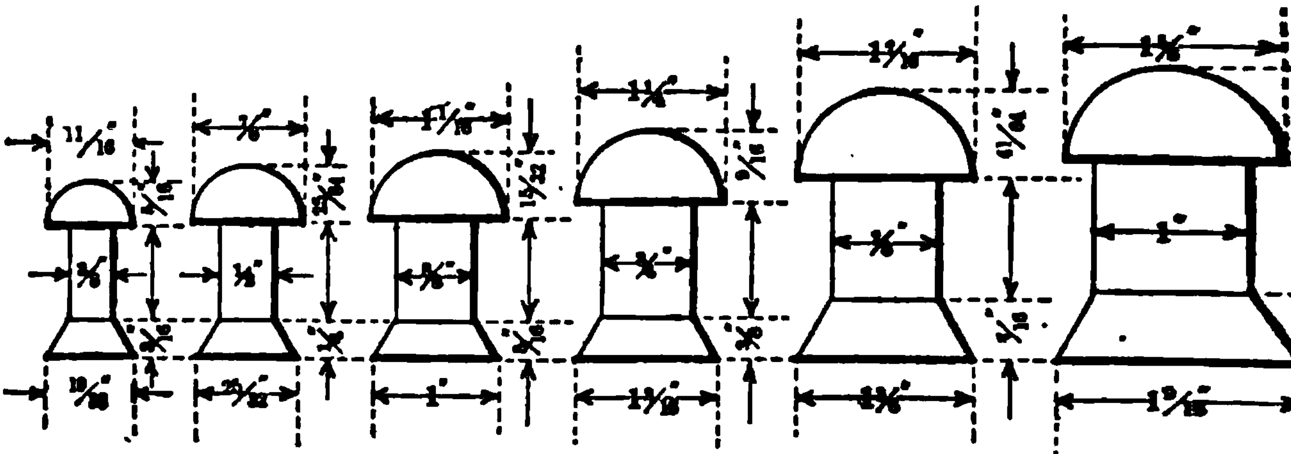
ables II and III give the SHEARING and BEARING VALUES for different sizes rivets in plates of different thickness for two values of working stresses each; r at 7 500 and 10 000 lb per sq in and bearing at 15 000 and 18 000 lb per in. Values for higher stresses can be figured by proportion from these tables. lower stresses should be used with wrought iron or in parts subjected to loads; the higher stresses where only constant or dead loads are present.

The SHEARING VALUE is equal to the area of the rivet multiplied by the work
stress; the BEARING VALUE is equal to the area of the projected surface un
pressure multiplied by the working stress in bearing, or, if

t = the thickness of the plate;
d = the diameter of the rivet;
and S_b = the working stress in bearing;
then the bearing value $P = dtS_b$

The Shearing and Bearing Values may be taken directly from the tab and if a rivet is in double shear, twice the quantity in the table is to be used its SHEARING VALUE. Quantities above the heavy broken lines are BEAR VALUES greater than the values in single shear, so that for these conditions, number of rivets necessary in a joint required to transmit a certain loa determined by dividing the load by the value in single shear. If rivets are double shear, the number of rivets required is found by dividing the load by BEARING VALUE.

Rivet-Proportions.* The following diagrams show various rivet-proporti including the dimensions of shanks and of finished and countersunk heads:



FINISHED HEADS

Diam head = 1½ diam of shank + ⅛" depth of head = 45/100 diam of head

COUNTERSUNK HEADS

Depth of head = ½ diam of shank. Bevel of head = 60°

* These proportions vary slightly at different mills and in different handbooks

Conventional Signs for Riveting. The following diagrams show some conventional signs for riveting:

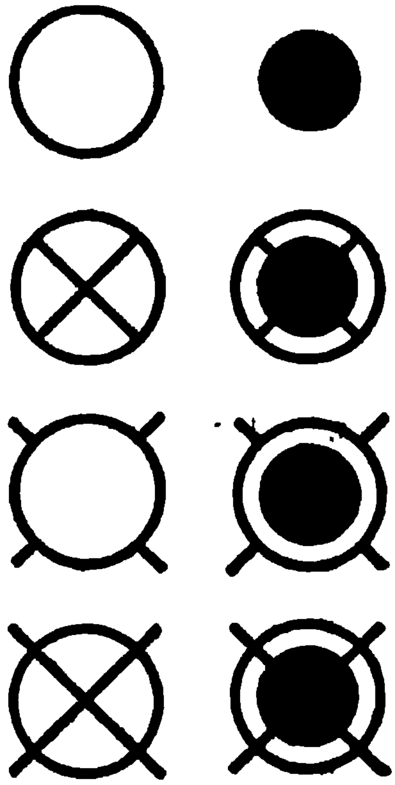
SHOP FIELD

Two Full Heads

Countersunk Inside (Farside) and Chipped

Countersunk Outside (Nearside) and Chipped

Countersunk both Sides and Chipped

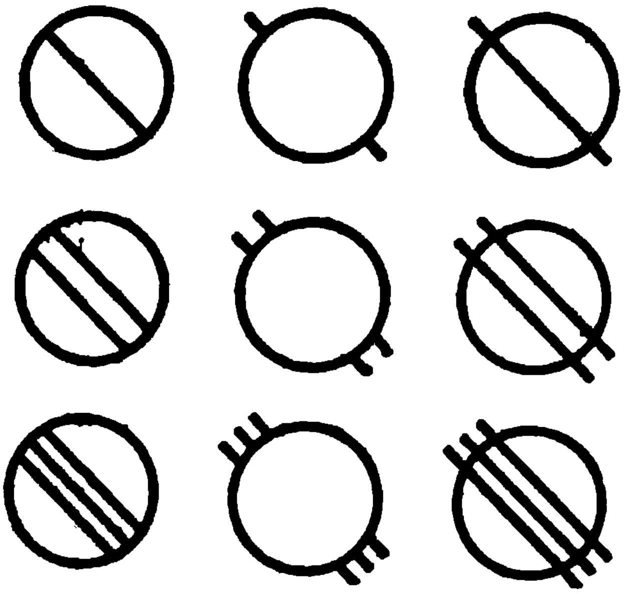


INSIDE (FARSIDE) OUTSIDE (NEARSIDE) BOTH SIDES

ttened to 1/8 in high or Countersunk and not Chipped

Flattened to 1/4 in high

Flattened 3/8 in high



s system, designed by F. C. Osborn, has for its foundation a diagonal cross present a countersink, a blackened circle for a field-rivet and a diagonal for a flattened head. The position of the cross with respect to the circle, outside, or on both sides, indicates the location of the countersink; and y, the number and position of the diagonal strokes indicate the height ition of the flattened heads. Any combination of field, countersunk ttened-head rivets liable to be used may be readily indicated by the bination of the above signs.

Table II. Shearing and Bearing Values* of Rivets
For Riveted Girders and Wrought Iron

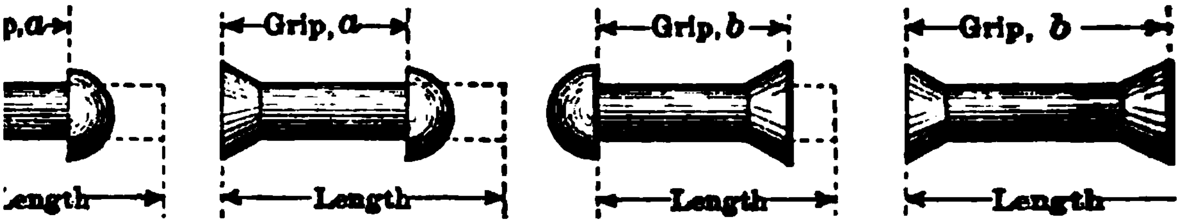
Diameter of rivet in inches		Area of rivet	Single shear at 7 500 lb per sq in	Bearing value for different thicknesses in inches of plate at 15 000 lb per sq in (=diameter of rivet × thickness of plate × 15 000 lb per sq in)										
Fractions	Decimals			1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8
3/8	0.375	0.1104	828	1 410	......	......	......	......	......	......	......	......	......	......
7/16	0.4375	0.1503	1 130	1 640	2 050	......	......	......	......	......	......	......	......	......
1/2	0.5	0.1963	1 470	1 880	2 340	2 810	......	......	......	......	......	......	......	......
9/16	0.5625	0.2485	1 860	2 110	2 640	3 160	3 690	......	......	......	......	......	......	......
5/8	0.625	0.3068	2 300	2 340	2 930	3 520	4 100	......	......	......	......	......	......	......
11/16	0.6875	0.3712	2 780	2 580	3 220	3 870	4 510	5 160	......	......	......	......	......	......
3/4	0.75	0.4418	3 310	2 810	3 520	4 220	4 920	5 630	6 330	......	......	......	......	......
13/16	0.8125	0.5185	3 890	3 050	3 810	4 570	5 330	6 090	6 860	7 620	......	......	......	......
7/8	0.875	0.6013	4 510	3 280	4 100	4 920	5 740	6 560	7 380	8 200	......	......	......	......
15/16	0.9375	0.6903	5 180	3 520	4 390	5 270	6 150	7 030	7 910	8 790	9 670	......	......	......
1	1.0	0.7854	5 890	3 750	4 690	5 620	6 560	7 500	8 440	9 380	10 310	11 250	......	......
1 1/16	1.0625	0.8866	6 650	3 980	4 980	5 980	6 970	7 970	8 960	9 960	10 960	11 950	12 950	......
1 1/8	1.125	0.9940	7 460	4 220	5 270	6 330	7 380	8 440	9 490	10 550	11 600	12 660	13 710	14 770
1 3/16	1.1875	1.1075	8 310	4 450	5 570	6 680	7 790	8 910	10 020	11 130	12 250	13 360	14 470	15 590

* Values for higher or lower unit stresses for shear or bearing to satisfy particular building laws can be figured by proportion from the table. See, [illegible] The Cambria Handbook gives values of from 12 000 to 20 000

										of plate at 18 000 lb per sq in plate × 18 000 lb per sq in)				
Fraction	Decimals	Area of rivet	Sin she 10 00 per s							5/8	11/16	3/4	13/16	7/8
3/8	0.375	0.1104	1 100	1 680	......	......	......	......	......	......	......	......	......	......
7/16	0.4375	0.1503	1 500	1 960	2 450	......	......	......	......	......	......	......	......	......
1/2	0.5	0.1963	1 960	2 240	2 800	3 370	......	......	......	......	......	......	......	......
9/16	0.5625	0.2485	2 480	2 520	3 150	3 790	4 420	......	......	......	......	......	......	......
5/8	0.625	0.3068	3 060	2 800	3 500	4 210	4 910	......	......	......	......	......	......	......
11/16	0.6875	0.3712	3 710	3 090	3 860	4 640	5 400	6 180	......	......	......	......	......	......
3/4	0.75	0.4418	4 420	3 370	4 210	5 060	5 890	6 740	7 580	......	......	......	......	......
13/16	0.8125	0.5185	5 180	3 650	4 560	5 480	6 390	7 300	8 210	9 120	......	......	......	......
7/8	0.875	0.6013	6 010	3 930	4 910	5 900	6 880	7 860	8 840	9 820	......	......	......	......
15/16	0.9375	0.6903	6 900	4 210	5 260	6 330	7 370	8 420	9 470	10 520	11 590	......	......	......
1	1.000	0.7854	7 850	4 500	5 600	6 750	7 860	9 000	10 120	11 240	12 370	13 500	......	......
1 1/16	1.0625	0.8866	8 860	4 780	5 970	7 170	8 360	9 560	10 750	11 940	13 140	14 340	15 530	......
1 1/8	1.125	0.9940	9 940	5 060	6 320	7 590	8 850	10 120	11 380	12 640	13 910	15 180	16 440	17 700
1 3/16	1.1875	1.1075	11 070	5 340	6 670	8 010	9 350	10 680	12 010	13 340	14 680	16 020	17 360	18 700

. Length of Field-Rivets for Various Grips. Length of Rivet-Sh to Form Head

American Bridge Company Standard. Dimensions in Inches



Diameter ½	Diameter ⅝	Diameter ¾	Diameter ⅞	Diameter 1	Grip b	Diameter ½	Diameter ⅝	Diameter ¾	Diameter ⅞
1½	1¾	1⅞	2	2⅛	½	1⅛	1¼	1¼	1⅜
1⅝	1⅞	2	2⅛	2¼	⅝	1¼	1⅜	1⅜	1½
1¾	2	2⅛	2¼	2⅜	¾	1⅜	1½	1½	1⅝
1⅞	2⅛	2¼	2⅜	2½	⅞	1½	1⅝	1⅝	1¾
2	2¼	2⅜	2½	2⅝	1	1⅝	1¾	1¾	1⅞
2⅛	2⅜	2½	2⅝	2¾	⅛	1¾	1⅞	1⅞	2
2¼	2½	2⅝	2¾	2⅞	¼	1⅞	2	2	2⅛
2⅜	2⅝	2¾	2⅞	3	⅜	2	2⅛	2⅛	2¼
2⅝	2⅞	3	3⅛	3¼	½	2⅛	2¼	2⅜	2⅜
2¾	3	3⅛	3¼	3⅜	⅝	2¼	2⅜	2½	2½
3	3¼	3⅜	3½	3⅝	¾	2½	2⅝	2¾	2¾
3⅛	3⅜	3½	3⅝	3¾	⅞	2⅝	2¾	2⅞	2⅞
3¼	3½	3⅝	3¾	3⅞	2	2¾	2⅞	3	3
3⅜	3⅝	3¾	3⅞	4	⅛	2⅞	3	3⅛	3⅛
3½	3¾	3⅞	4	4⅛	¼	3	3⅛	3¼	3¼
3⅝	3⅞	4	4⅛	4¼	⅜	3⅛	3¼	3⅜	3⅜
3¾	4	4⅛	4¼	4⅜	½	3¼	3⅜	3½	3½
3⅞	4⅛	4¼	4⅜	4½	⅝	3⅜	3½	3⅝	3⅝
4	4¼	4⅜	4½	4⅝	¾	3½	3⅝	3¾	3¾
4⅛	4⅜	4½	4⅝	4¾	⅞	3⅝	3¾	3⅞	3⅞
4⅜	4⅝	4¾	4⅞	5	3	3⅞	4	4	4⅛
4½	4¾	4⅞	5	5⅛	⅛	4	4⅛	4⅛	4¼
4⅝	4⅞	5	5⅛	5¼	¼	4⅛	4¼	4¼	4⅜
4¾	5	5⅛	5¼	5⅜	⅜	4¼	4⅜	4⅜	4½
4⅞	5⅛	5¼	5⅜	5½	½	4⅜	4½	4½	4⅝
5	5¼	5⅜	5½	5⅝	⅝	4½	4⅝	4⅝	4¾
5⅛	5⅜	5½	5⅝	5¾	¾	4⅝	4¾	4¾	4⅞
5¼	5½	5⅝	5¾	5⅞	⅞	4¾	4⅞	4⅞	5
5⅜	5⅝	5¾	5⅞	6	4	4⅞	5	5	5⅛
5½	5⅞	6	6⅛	6¼	⅛	5⅛	5¼	5¼	5⅜
5¾	6	6⅛	6¼	6⅜	¼	5¼	5⅜	5⅜	5½
6	6¼	6⅜	6½	6⅝	⅜	5½	5⅝	5⅝	5⅝
6⅛	6⅜	6½	6⅝	6¾	½	5⅝	5¾	5¾	5¾
6¼	6½	6⅝	6¾	6⅞	⅝	5¾	5⅞	5⅞	5⅞
6⅜	6⅝	6¾	6⅞	7	¾	5⅞	6	6	6
6½	6¾	6⅞	7	7⅛	⅞	6	6⅛	6⅛	6⅛
6⅝	6⅞	7	7⅛	7¼	5	6⅛	6¼	6¼	6¼
....	....	7⅛	7¼	7⅜	⅛	....	....	6⅜	6⅜
....	....	7¼	7⅜	7½	¼	....	....	6½	6½
....	....	7⅜	7½	7⅝	⅜	....	....	6⅝	6⅝
....	....	7⅝	7¾	7⅞	½	....	....	6⅞	6⅞
....	....	7¾	7⅞	8	⅝	....	....	7	7
....	....	7⅞	8	8⅛	¾	....	....	7⅛	7⅛
....	....	8	8⅛	8¼	⅞	....	....	7¼	7¼

For weight of rivets, see page 1528.

…e of Riveted Joints. Riveted joints are used in building-construction (1) in tie-bar splices, (2) in floor-beam connections, (3) in the joints of trusses, (4) in riveted girders, and (5) in column-connections.

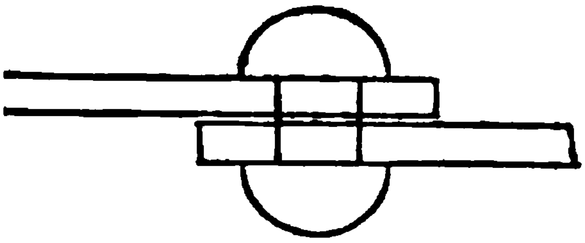


Fig. 10. Lap-joint

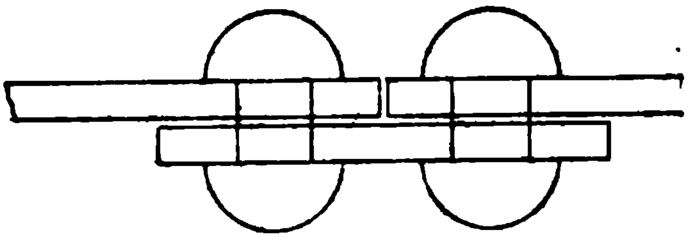


Fig. 11. Butt-joint with Single Cover-plate

…licing of Tie-Bars. Tie-bars may be spliced by a LAP-JOINT (Fig. 10); …a BUTT-JOINT with a single cover-plate (Fig. 11); or by a BUTT-JOINT …h two cover-plates (Fig. …).

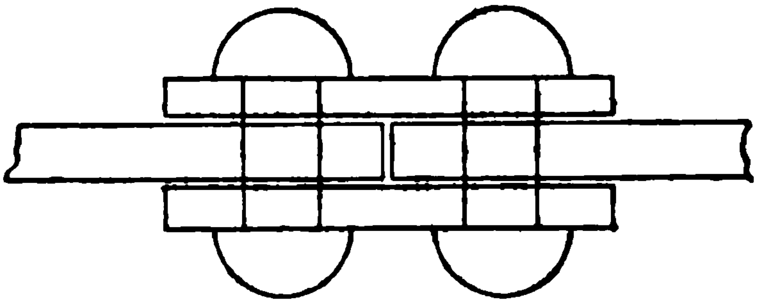
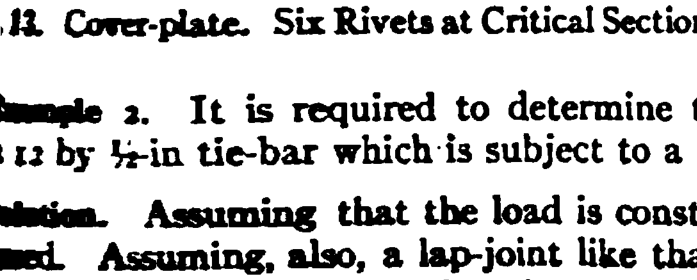


Fig. 12. Butt-joint with Two Cover-plates

…he Butt-Joint is symmet…l and more efficient than …others because of the absence …y tendency to bend when …er a load. The net area of …cover-plates at the section …ugh the rivets at the end of …main plate must be equal to …net area of the main plate through the rivets at the end of the cover-plate. … 14 shows a better arrangement of rivets than that in Fig. 13, because less … is removed at the critical section of the cover-plates. In some cases it may be necessary to make the aggregate thickness of the cover-plates greater than the thickness of the main plates.



…13. Cover-plate. Six Rivets at Critical Section

A joint with one line of rivets is said to be SINGLE-RIVETED, one with two lines DOUBLE-RIVETED, and one with more than two lines, CHAIN-RIVETED.

…ample 2. It is required to determine the number of rivets in the splice … 12 by ½-in tie-bar which is subject to a tensile force of 65 000 lb.

…ution. Assuming that the load is constant, the stresses in Table III may …ed. Assuming, also, a lap-joint like that in Fig. 15, and ¾-in rivets, the …e in shear of one rivet is found to …4 420 lb and the bearing value …nst a ½-in plate, 6 740 lb. The …ber of rivets is determined by the …r to be equal to 65 000 divided by …0, or fifteen. Since sixteen rivets … required to complete a figure …ller but similar in arrangement to … shown in Fig. 15, this number is …. There is some latitude possible in the spacing of the rivets, but with a …h of 12 in, the horizontal gauge-lines are placed 1½ in apart for symmetry. …e pitch P, as shown in Fig. 15, is required to be three times the diameter …e rivet, this diagonal pitch across the rivet-spacing must be 2.25 in, or

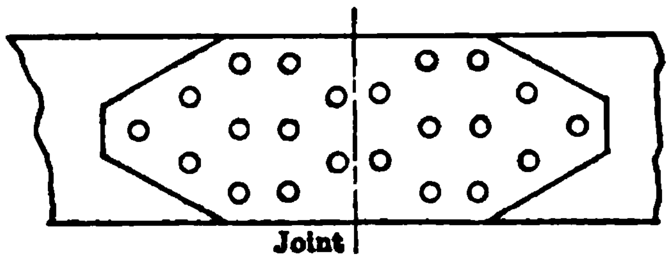


Fig. 14. Cover-plate. Four Rivets at Critical Section

greater. The length of the horizontal or third side of the right-angled trian having an hypotenuse of 2.25 in and a vertical altitude of 1.5 in, is 1.68 which requires that this dista *ED*, etc., be 1.75 in, if measu in multiples of ¼ in.

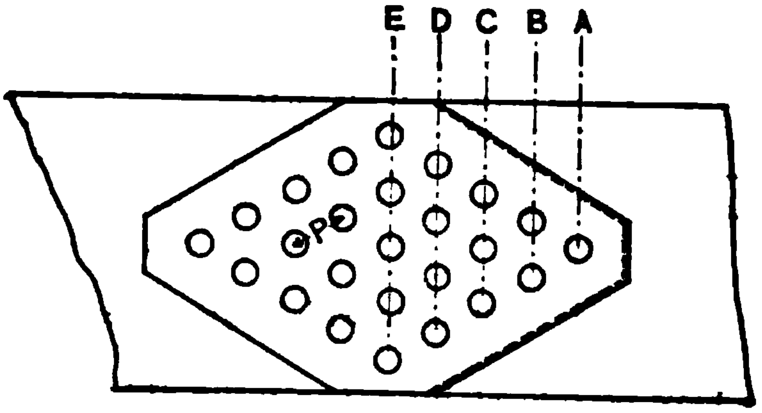


Fig. 15. Rivet-spacing in Cover-plate

Floor-Beam Connectio The two following examples lustrate common types of fl beam connections.

Example 3. It is required determine the number of ¾ rivets to connect a 10-in 25.4 I beam supporting 24 000 lb a 15-in 42.9-lb I beam, usin shearing-stress of 10 000 lb per sq in and a bearing-stress of 18 000 per sq in.

Solution. From the table of properties of standard I beams, pages 354–5, thickness of the web of the 10-in 25.4-lb beam is found to be 0.31 in, say 5⁄16 and of the 15-in 42-lb beam, 0.41 in, say 7⁄16 in. Referring to Table III, p 419, the bearing values for a ¾-in rivet for these thicknesses of webs are spectively 4 210 lb and 5 890 lb. The shearing value of the rivet is 4 420 The rivets in the 10-in beam are in double shear; hence the bearing value g erns. The number of rivets, then, is 12 000, the end-reaction, divided by 4 or 3. For the 15-in beam the shearing value is less, and the number of ri required is 12 000 divided by 4 420, or 3. Hence two standard connect angles, 6 by 4 by ⅝ in and 5 in long, may be used. Each has three holes in leg and two in the other. The leg with three holes is placed on the 10-in b with the rivets in double shear, and the leg with two holes is connected to 15-in beam; thus, in the latter case there are four rivets where only three required for strength. They are driven in the field during the erection of structure and the working stress is accordingly made less in most specificat because of the better work possible with the heavy machines used in shop-w than with the tools available in the field.

Example 4. It is required to determine the number of ¾-in rivets i 4 by 4 by ½-in angle-bracket attached to an 18-in 54.7-lb beam and supp ing a 10 by 12-in wooden beam on which there is a load of 18 000 lb.

Solution. The rivets are in single shear with a shearing-resistance of 4 42 taken from Table III. The thickness of the web of the I beam is 7⁄16 in, gi a bearing value of 5 890 lb. Dividing 9 000 lb, the end-reaction, by 4 42 the controlling value, we find that two rivets are insufficient. The bracket be fastened with three ¾-in rivets with a spacing of 4 in. Two ⅞-in ri are sufficient to hold the bracket.

Rivets in Plate Girders. The methods of determining the rivets in p and box girders are given in Chapter XX.

Bending Stress in Rivets. While the BENDING STRENGTH of PINS at joints of articulated trusses is always investigated, this is never done in case of RIVETS. A hot rivet properly driven is, when cold, under a te stress which is nearly equal to the elastic limit of the material. This c great pressure between the plates and a consequent frictional resistanc movement, which, under the usual conditions, equals the allowed shearing- on the rivet; and so, until an INITIAL SLIP occurs, there can be no BEN STRESSES in the rivet. In the case of very long rivets driven in holes w

ere is an imperfect alinement of the plates and a consequent difficulty in aking the rivets fill the holes completely, it is not probable that any large ading stresses can occur in the rivets of a structure. This has been avoided n a few structures for which long TAPER RIVETS were specified to be used in oles REAMED with TAPERED REAMERS, thus insuring a perfect filling of the holes.

Working Stresses. Tables II and III are based on stresses which approximate those used in the best practice. Table II is used for the few structures ade of WROUGHT IRON and for those places in steel structures that are subject o severe conditions of service, as in the floor-systems of bridges. Table III s used for ordinary structural work made under the conditions governing in odern shop-practice. For comparison, the following stresses taken from the pecifications of Theodore Cooper for Steel Railroad Bridges and Steel Highway ridges are given:

Specification for	Allowable stresses on rivets, lb per sq in	
	Bearing	Shear
Steel railroad bridges	15 000 (12 000 on floors) 22 500 for laterals	9 000 (7 200 on floors) 13 500 for laterals
Steel highway bridges	18 000 (14 400 on floor-beams) 27 000 for laterals	10 000 (8 000 on floor-beams) 14 000 for laterals

Rivets driven in the field are allowed two-thirds the value of shop-driven rivets.

2. Strength of Pins in Trusses*

Truss-Pins. In the design of the PINS at the joints of trusses the stresses in HEAR, BEARING FLEXURE or BENDING must be investigated.

The Shearing-Force at any section of the pin is the algebraic sum of all e forces acting on the pin on either side of the section. The stress is considered o be uniformly distributed over the cross-section of the pin. When the forces o not act in the same plane they must be resolved into vertical and horizontal mponents and the resultant of these components taken as the shear at any sired section. This may be done by the principles of GRAPHIC STATICS, or y TRIGONOMETRICAL and ALGEBRAICAL METHODS, the graphic method being, r some, the more rapid.

The Bearing Area on the pin is taken as the PROJECTION OF THE AREA OF NTACT, the area of this projection being equal to the diameter of the pin ultiplied by the thickness of the plate. The bearing is assumed to be uniformly distributed; hence for any load the intensity of the pressure may be creased by increasing the thickness of the plate or the diameter of the pin.

The Bending Moments on the pin may be found by the PRINCIPLE OF MENTS or by methods involving the principles of GRAPHIC STATICS explained Chapter IX in finding the bending moments of beams. The forces are coniered to be concentrated at the middle of the bearing-plates. If they do not e in a plane with the pin they must be resolved into their vertical and hori-

* Since the introduction of rolled-steel shapes and riveted joints, pin-joints for trusses f moderate span in buildings have fallen into disuse. The general principles of their sign, however, are given here.

zontal components and these component forces in the two planes treated sep rately. The resultants in both planes at any section may be combined and single resultant force acting on the section obtained, and also the conseque stresses due to it.

Table V. Shearing and Bearing Values of Pins for One-Inch Thickness of Plate, in Pounds per Square Inch

Diameter of pin, in	Area of pin, sq in	Bearing value at 12 000 lb per sq in, lb	Single shear 7 500 lb per sq in, lb	Diameter of pin, in	Area of pin, sq in	Bearing value at 12 000 lb per sq in, lb	Single shear 7 5 lb per sq i tons
1	0.785	12 000	5 890	4	12.57	48 000	47.0
1⅛	0.994	13 500	7 455	4⅛	13.36	49 500	50.1
1¼	1.227	15 000	9 202	4¼	14.19	51 000	53.2
1⅜	1.485	16 500	11 132	4⅜	15.03	52 500	56.3
1½	1.767	18 000	13 252	4½	15.90	54 000	59.6
1⅝	2.074	19 500	15 555	4⅝	16.80	55 500	63.0
1¾	2.405	21 000	18 037	4¾	17.72	57 000	66.3
1⅞	2.760	22 500	20 707	4⅞	18.67	58 500	70.0
2	3.142	24 000	23 565	5	19.64	60 000	73.6
2⅛	3.547	25 500	26 600	5⅛	20.63	61 500	77.3
2¼	3.976	27 000	29 820	5¼	21.65	63 000	81.2
2⅜	4.430	28 500	33 225	5⅜	22.69	64 500	85.1
2½	4.909	30 000	36 817	5½	23.76	66 000	89.1
2⅝	5.412	31 500	40 590	5⅝	24.85	67 500	93.2
2¾	5.940	33 000	44 550	5¾	25.97	69 000	97.3
2⅞	6.492	34 500	48 690	5⅞	27.11	70 500	101.1
			tons				
3	7.069	36 000	26.5	6	28.27	72 000	106
3⅛	7.670	37 500	28.7	6⅛	29.46	73 500	110
3¼	8.296	39 000	31.0	6¼	30.68	75 000	115
3⅜	8.946	40 500	33.5	6⅜	31.92	76 500	119
3½	9.621	42 000	36.0	6½	33.18	78 000	124
3⅝	10.32	43 500	38.7	6⅝	34.47	79 500	129
3¾	11.05	45 000	41.4	6¾	35.79	81 000	134
3⅞	11.79	46 500	44.2	6⅞	37.12	82 500	139

In the Method of Moments a section is taken at each force in success and the moment of the forces about a point in the section found, due consid ation being given to the di tion of turning. This is d at each force on one side of pin, if the bars are arran symmetrically, and in both vertical and horizontal pla Inspection of the results usually indicate which sec has the GREATEST RESULTANT MOMENT when the horizontal and vert components, H and V, are combined. This is done by using the form $R^2 = H^2 + V^2$ since, graphically, the resultant R is the diagonal of the recta

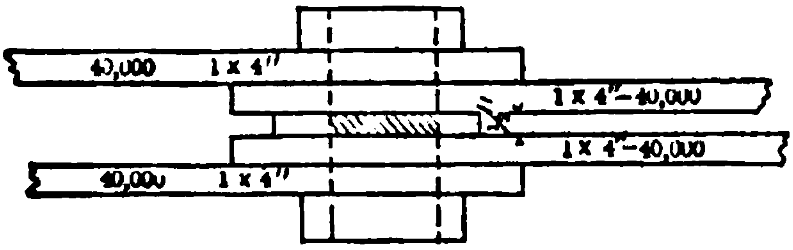


Fig. 16. Pin-joint

Table VI. Maximum Bending Moments in Inch-Pounds to be Allowed on Pins for Maximum Fiber-Stresses of 15 000, 20 000 and 22 500 Pounds per Square Inch

Diameter of pin, in	Moment for $S =$ 15 000 in-lb	Moment for $S =$ 20 000 in-lb	Moment for $S =$ 22 500 in-lb	Diameter of pin, in	Moment for $S =$ 15 000 in-lb	Moment for $S =$ 20 000 in-lb	Moment for $S =$ 22 500 in-lb
1	1 470	1 960	2 210	4	94 200	125 700	141 400
1⅛	2 100	2 800	3 140	4⅛	103 400	137 800	155 000
1¼	2 880	3 830	4 310	4¼	113 000	150 700	169 600
1⅜	3 830	5 100	5 740	4⅜	123 300	164 400	185 000
1½	4 970	6 630	7 460	4½	134 200	178 900	201 300
1⅝	6 320	8 430	9 480	4⅝	145 700	194 300	218 500
1¾	7 890	10 500	11 800	4¾	157 800	210 400	236 700
1⅞	9 710	12 900	14 600	4⅞	170 600	227 500	255 900
2	11 800	15 700	17 700	5	184 100	245 400	276 100
2⅛	14 100	18 800	21 200	5⅛	198 200	264 300	297 300
2¼	16 800	22 400	25 200	5¼	213 100	284 100	319 600
2⅜	19 700	26 300	29 600	5⅜	228 700	304 900	343 000
2½	23 000	30 700	34 500	5½	245 000	326 700	367 500
2⅝	26 600	35 500	40 000	5⅝	262 100	349 500	393 100
2¾	30 600	40 800	45 900	5¾	280 000	373 300	419 900
2⅞	35 000	46 700	52 500	5⅞	298 600	398 200	447 900
3	39 800	53 000	59 600	6	318 100	424 100	477 100
3⅛	44 900	59 900	67 400	6⅛	338 400	451 200	507 600
3¼	50 600	67 400	75 800	6¼	359 500	479 400	539 300
3⅜	56 600	75 500	84 900	6⅜	381 500	508 700	572 300
3½	63 100	84 200	94 700	6½	404 400	539 200	606 600
3⅝	70 100	93 500	105 200	6⅝	428 200	570 900	642 300
3¾	77 700	103 500	116 500	6¾	452 900	603 900	679 400
3⅞	85 700	114 200	128 500	6⅞	478 500	638 000	717 800

Remarks. The following is the formula for flexure, $M = SI/c$, with the reductions made to adapt it to a beam of circular section:

$$M = S\pi d^3/32 = SAd/8$$

M = the moment of forces for any section through the pin;
S = the stress per sq in in extreme fibers of pin at that section;
A = the area of the section;
d = the diameter;
π = 3.14159.

The forces are assumed to act in a plane passing through the axis of the pin.
The above table gives the values of M for different diameters of pin, and for three values of S.
If the maximum value of M is known, an inspection of the table will show what the diameter of the pin must be so that S will not exceed 15 000, 20 000, or 22 500 lb, as the requirements of the case may be.

H and V. Example 6 illustrates the method for the condition of INCLINED forces acting on the pin. In Example 5 the same method is employed to determine the size of the pin in a simple joint.

Example 5. It is required to determine the size of the pin for the joint sho in Fig. 16 in the lower chord of a steel truss. The middle bar is a vertical s pension-rod to hold the chord in place.

Solution. Beginning at the section between the outer bars, the algebr sum of the forces on either side of the section is 40 000 lb, hence this is t shear. At the section next to the suspender the sum is zero; therefore th is no shear at the middle of the pin. The bearing pressure is 40 000 lb. intensity depends on the diameter of the pin and the thickness of the bars. find the bending moment on the pin the forces are considered concentrated the middle of the bars and moments taken about sections through the forc The moment at the section through the second bar is 40 000 lb × 1 in, eq to 40 000 in-lb. If moments are taken about a point between the inner for the same result is obtained. From Table VI it is found that a 2¾-in pin 20 000 lb per sq in is sufficient. From Table V the bearing value of a 2¾ pin is found to be only 33 000 lb at a stress of 12 000 lb per sq in, which mal it necessary to increase the size of the pin to 3⅝ in. The shearing value this pin is 67 000 lb. In this case the diameter of the pin is determined the bearing-stress, but it is necessary to investigate the other stresses to sure of the correct size, especially in case of heavy bearing-plates.

Bending Moments on Pins. The finding of the BENDING MOMENT due the forces acting on a pin is usually the most difficult part of the work of det mining its proper size. In the case of a simple pin, properly packed and lyi in the plane of the forces acting on it, the GREATEST MOMENT is usually the pr uct obtained by multiplying the outer force by the central distance betw the outer bars; but when the forces act in several planes the work is m complicated. The GRAPHICAL METHOD illustrated in the solution of the t following examples has some advantages; but the METHOD OF MOMENTS appl at the end of the solution of the first example is equally rapid in practi hands and capable of greater refinement in the results.

Example 6. It is required to find the bending moment on the pin of t joint, one-half of which is shown in Fig. 17. The bars are each 1 in thi the channel of the vertical member ½ in thick and the center of the han is ¾ in from the center of the channel.

Solution. Since the joint is symmetrical it is necessary to construct but o half of the force-diagram and equilibrium-polygon which really apply the joint. From the conditions of equilibrium of forces, the vertical co ponent of the inclined force is upward, and equal to the sum of the downw forces, 34 000 lb; and its horizontal component acts with the 60 000-lb for to the amount of 17 000 lb, a sufficient amount to close the force-diagra The following construction is special, in that but one-half of the entire graphi diagram is shown. This is made possible because of the symmetry of the joi the bending moment being constant over the middle of the pin.

In the diagram (Fig. 18) AB is drawn at an angle of 45° with the horizon and commencing at c, the distances are laid off to scale between the bars, the lines 1–2, 2–3, etc., drawn parallel to the forces they represent at the joi The oblique force is resolved into its components 1–4 and 1–5.

The stress-diagram (Fig. 19) is drawn as follows: On a horizontal line forces are laid off to scale in the order they occur on the pin, 1–2, 2–3, 3–4 4–1, the closing of the diagram being a check on the correctness of the valu the forces. Beginning at 1, 1–5, 5–6 and 6–1 are laid off to scale, paralle the forces in the vertical plane. From 1 the line 1–0 is drawn at an angle 45°, for convenience in making good intersections, and equal to a conveni number, say 20 000 lb, in the same scale to which the loads are drawn. T

at O is the pole of the stress-diagram, the pole-distance being 20 000 lb. From the principles of graphics the bending moment at any point on the pin equal to the intercept between the proper ray of the equilibrium-polygon and the closing line, multiplied by the pole-distance. To complete the figures, o-3 and o-4 are drawn from O, and from c cd is drawn parallel to o-2, de parallel to o-3, ef parallel to o-4 and fk parallel to o-1. In the same way is drawn parallel to o-5, st to o-6 and tn to o-1. Then according to the above principles, the moment at any section due to the forces in the horizontal plane is proportional to the ordinate at that section drawn from the line AB the line $cdefk$ bounding the equilibrium-polygon; and the moments due to the vertical forces are proportional to the ordinates drawn to the line $rstn$, the numerical value being the length of the ordinate times 20 000, the pole-distance.

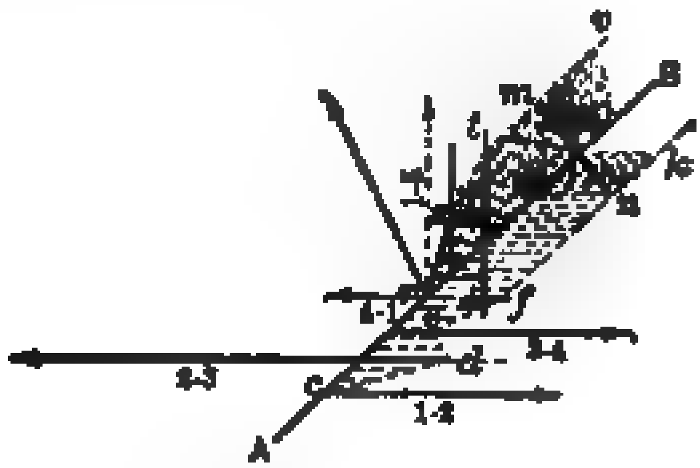


Fig. 18

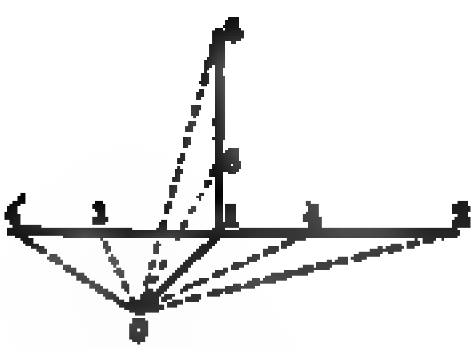


Fig. 19

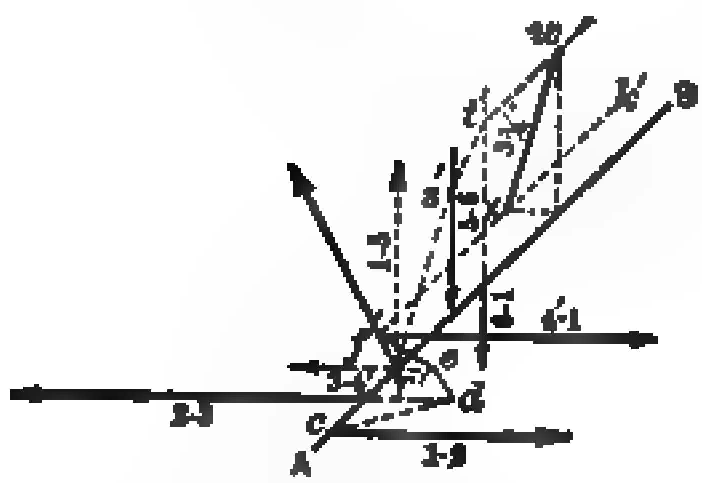


Fig. 20

Figs. 17 to 20. Pin-joint and Moment-diagrams

...re both moments are present, the resultant or true moment is proportional ...he hypotenuse of the right-angled triangle having for its sides the ordinates ...he two planes at the point in question. At X this is shown by the line mn. ...measures 2.42 in, and being the longest diagonal or hypotenuse that can ...rawn in the figure, it follows that the maximum bending moment on the ...is $2.42 \times 20\,000 = 48\,400$ in-lb.

...o find the effect of changing the arrangement of the members on the pin, ...may be assumed that the inclined bar is placed outside the inner chord-bar. ...horizontal stress-diagram then becomes 1-2, 2-3, 3-4′, 4′-1. The equilibrium-polygons become $cdef'k'$ and $r's't'w$, as shown in Fig. 20. In these polygons ...longest diagonal measures 3¾ in, which gives a bending moment of 3¾ ...20 000 lb = 75 000 in-lb, showing that the arrangement of the eye-bars in ...17 is better. As a rule the bending moment is less when those forces that

oppose each other are placed together. It may be further reduced by ma
the outside bar one-half the thickness of the main horizontal bars.

To check by the METHOD OF MOMENTS the value of the maximum ben
moment obtained by the GRAPHIC METHOD for the first arrangement, the
ments of the forces in the horizontal plane are taken about r. This gives

$$M_h = 38\,500 \text{ lb} \times 3.0 \text{ in} + 38\,500 \text{ lb} \times 1.0 \text{ in} - 60\,000 \text{ lb} \times 2.0 \text{ in}$$
$$= 34\,000 \text{ in-lb},$$

which is the value of the moment in the horizontal plane across the midd
the pin.

In the vertical plane moments are taken about a point t, giving

$$M_v = 34\,000 \text{ lb} \times 1.5 \text{ in} - 22\,000 \text{ lb} \times 0.75 \text{ in}$$
$$= 34\,500 \text{ in-lb}$$

From these component moments the resultant maximum bending moment

$$M = \sqrt{34\,000^2 + 34\,500^2} = 48\,400 \text{ in-lb}$$

Example 7. Another illustration of the GRAPHICAL METHOD of finding
bending moment on a pin is given for the joint A of the truss-diagram show

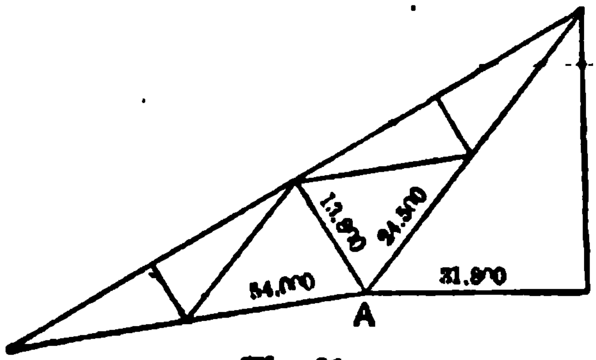


Fig. 21

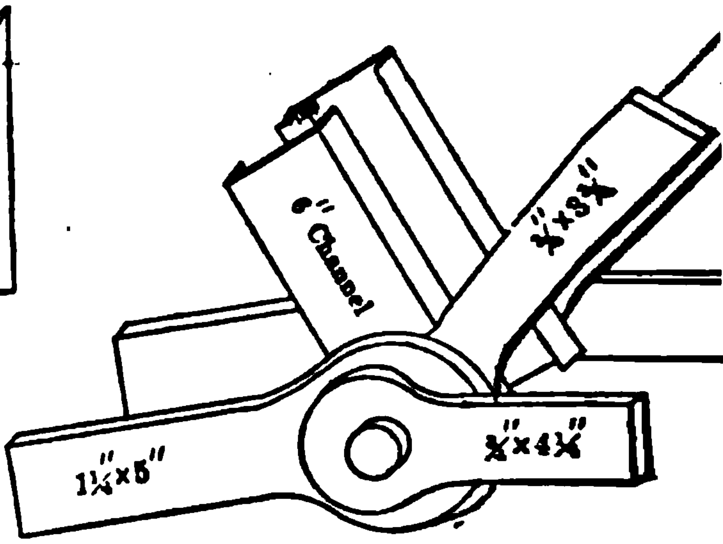


Fig. 22

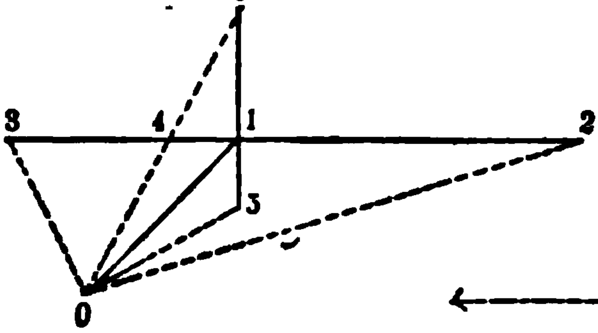


Fig. 23

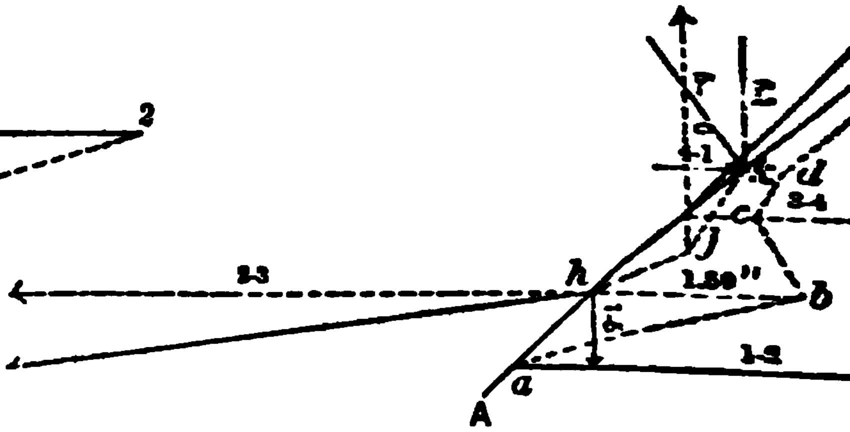


Fig. 24

Figs. 21 to 24. Force-polygons and Equilibrium-polygons for Bending Moment a Pin

Fig. 21. Fig. 22 shows the arrangement and size of the members. The s
given in Fig. 21 are for one-half the number of members at the joint.
Example 6, the symmetrical arrangement makes it unnecessary to draw

1 one-half of the force-polygon and equilibrium-polygon. The web of the nd is reinforced to make it ⅝ in thick.

lution. The line AB (Fig. 24) is drawn at an angle of 45° and ah, etc., are off to scale, equal to the distances between the members. At each point plication of a force a line is drawn parallel and to scale, to represent that e. The inclined forces are then resolved into their horizontal and vertical ponents. The force-diagram (Fig. 23) is then drawn, the horizontal forces g laid off to scale in the order in which they occur, 1–2, 2–3, 3–4 and 4–1. pole-distance is then laid off at an angle of 45° and equal to 20 000 lb to same scale of forces. The pole o is then joined with 2, 3, 4 and 1. Then Fig. 24, ab is drawn parallel to o–2, bc to o–3, cd to o–4 and de to o–1. In same way the line $hjkB$ is drawn. From inspection it is seen that hb is the est intercept, even longer than any diagonal that may be drawn from the emities of the horizontal and vertical intercepts at any point along AB. the same scale that makes o–1 represent 20 000 lb, hb represents 31 800 in-lb; efore the bending moment on the pin is 31 800 in-lb. In Table VI a pin in in diameter, at a fiber-stress of 20 000 lb per sq in, has an allowable ment of 35 500 in-lb, and in Table V a bearing value on 1 in of 31 500 lb. ure of 31 800 lb on ¾ in is equal to 42 400 for a 1-in bar; so it is necessary e a larger pin to accommodate the bearing requirement. From Table V n 3½ in in diameter is found to be necessary. The shearing value of this is 72 000 lb more than twice the load, so, again, it is the bearing that con- s the size of the pin. If the thickness of the bars is increased the diameter he pin may be reduced to 3 in.

4. Strength of Bolts in Wooden Trusses and Girders

he Working Stresses for Bolts on which Table VII and Table VIII are puted are based on a FACTOR OF SAFETY of five applied to the average of y tests on dry timber. In some specifications it is permitted to increase the ING PRESSURE between timber and bolts as much as 50% above that per- ted for short struts. The values in the tables are somewhat less than the s on large trusses made at the Massachusetts Institute of Technology, in h would indicate as safe values. These were reported in the Engineering rd, November 17, 1900. Table IX gives the allowable maximum tension, r and bending moments for wrought-iron and steel bolts.

nds of Stress in Bolts. BOLTS in wooden trusses are subject to the e kinds of stress as the RIVETS and PINS in steel structures. When the s joined are less than 2 in thick and the bolts are tightly drawn up so as evelop considerable frictional resistance between the pieces, the bolts are rtioned to resist the total force in SHEAR and in BEARING. When the s are more than 2 in thick the BENDING is taken into account and the bolts t be investigated for stresses in SHEAR, in BEARING and in BENDING. The r is assumed to be uniformly distributed over the cross-section of the bolt, the BEARING AREA is the area of the projection of the bolt on the timber, h area is equal to the diameter of the bolt multiplied by the length in con- . The BEARING STRENGTH is given as a property of the bolt although its p depends upon the crushing strength of the timber. The BENDING MO- T on the bolt is found in the same manner as for pins in steel trusses, ugh the cases are usually less complicated.

strations of the Use of Bolts. The principles involved in the use of in wooden trusses and girders and in the use of the tables may be best ated by the solution of examples in each of the following cases:

(1) Bolts in tie-beams, thin pieces.
(2) Bolts in girders to support brackets.
(3) Bolts as pins in the joints of trusses.
(4) Bolt-and-strap joints in trusses.
(5) Bolts under tension to hold the foot of a rafter.

(See, also, "Joints in Wooden Trusses," Chapter XXVIII, pages 1149 to

Case 1. Bolts in Tie-Beams, Thin Pieces. Tie-beams of wooden tr when longer than 30 ft, are usually made up of a number of pieces. This struction is cheaper than the use of a single stick. Two-inch planks b together are generally used. The location of the joints in the courses of p and the number and size of the bolts are the special considerations in the d of such a joint. In general, the joints in adjacent courses are placed a apart as possible and not more than two joints are placed opposite each in the same section. The simplest case is that of a plain FISH-PLATE JOIN a common BUTT-JOINT with two cover-plates as shown in Fig. 12. The nu of BOLTS for such a joint is found in the same way as the number of RIVE steel tie-bars. The bolts must be spaced as required in the second column each timber in Table VII, to provide against shearing in front of the bolt.

Table VII.* Safe Bearing Value of Bolts per Inch of Length Parallel the Grain in Timber and Distance from Center to Center of Bolts or to End of Timber

Diameter of bolt, in	Long-leaf yellow pine		White pine and short-leaf yellow pine		Douglas fir		White o
	Bearing at 1 400 lb per sq in, lb	Distance, in	Bearing at 1 100 lb per sq in, lb	Distance, in	Bearing at 1 200 lb per sq in, lb	Distance, in	Bearing at 1 400 lb per sq in, lb
3/4	1 050	4 1/2	825	5 1/4	900	4 1/4	1 050
7/8	1 225	5	960	5 3/4	1 050	5	1 225
1	1 400	5 3/4	1 100	6 1/2	1 200	5 1/2	1 400
1 1/8	1 575	6 1/2	1 237	7 1/2	1 350	6 1/4	1 575
1 1/4	1 750	7	1 375	8	1 500	7	1 750
1 3/8	1 925	7 3/4	1 512	9	1 650	7 3/4	1 925
1 1/2	2 100	8 1/2	1 650	9 3/4	1 800	8 1/2	2 100
1 3/4	2 450	10	1 925	11 1/2	1 950	9 1/4	2 450
2	2 800	11 1/2	2 200	13	2 400	11 1/4	2 800
2 1/4	3 150	12 3/4	2 475	14 3/4	2 700	12 1/2	3 150
2 1/2	3 500	14 1/4	2 750	16 1/4	3 000	14	3 500
2 3/4	3 850	15 1/4	3 025	18	3 300	15 1/2	3 850
3	4 200	17	3 300	19	3 600	17	4 200

The distance from the end is equal to the diameter of the bolt plus the length o twice the SHEAR is equal to the BEARING VALUE of the bolt against the end-fibe Notes with Table XVI, Chapter XVI, for increase in allowed stresses.

* When the effect of the inclined surfaces upon the unit stresses is taken into the formula for the normal intensity of stress for cylindrical pins or bolts, given in XXVIII, page 1138, may be used. This formula will give lower values than Table VII.

ble VIII.* Safe Bearing Value of Bolts per Inch of Length Across the Grain in Timber

Diameter of bolt, in	Long-leaf yellow pine, lb	Short-leaf yellow pine and Douglas fir, lb	White pine, lb	White oak, lb
¾	262	187	150	375
⅞	306	218	175	437
1	350	250	200	500
1⅛	394	281	225	562
1¼	437	312	250	625
1⅜	482	343	275	687
1½	525	375	300	750
1¾	612	437	350	875
2	700	500	400	1000

ble IX. Maximum Allowable Tension, Shear and Bending Moment for Wrought-Iron and Steel Bolts

...meter of bolt, in	Net area, sq in	Wrought iron			Steel		
		Tension at 12 000 lb per sq in, lb	Shear at 7 500 lb per sq in, lb	Bending moment at 15 000 lb per sq in, in-lb	Tension at 16 000 lb per sq in, lb	Shear at 10 000 lb per sq in, lb	Bending moment at 20 000 lb per sq in, in-lb
¾	0.302	3 620	3 310	620	4 830	4 420	830
⅞	0.420	5 040	4 510	980	6 720	6 010	1 310
1	0.550	6 600	5 890	1 470	8 800	7 850	1 960
1⅛	0.694	8 328	7 460	2 100	11 100	9 940	2 800
1¼	0.893	10 716	9 200	2 880	14 290	12 270	3 830
1⅜	1.057	12 680	11 140	3 830	16 910	14 850	5 100
1½	1.295	15 540	13 250	4 970	20 720	17 670	6 630
1¾	1.746	20 930	18 040	7 890	27 910	24 050	10 500
2	2.302	27 620	23 560	11 800	36 830	31 420	15 700
2¼	3.023	36 280	29 820	16 800	48 370	39 760	22 400
2½	3.719	44 630	36 820	23 000	59 510	49 090	30 700
2¾	4.620	55 430	44 550	30 600	73 910	59 400	40 800
3	5.428	65 140	53 010	39 800	86 850	70 690	53 000
3¼	6.510	78 120	62 220	50 600	104 160	82 960	67 400

mple 8. A typical tie-beam used as a lower chord of a Howe truss i n in Fig. 25. It is 50 ft long, of Douglas fir and subject to the tension in ifferent panels shown in the figure.

ution. The thickness of the plank is drawn out of scale in the figure to the joints more clearly. The black circles show the vertical tension-rods, h so nearly cut the middle plank in two that it is not considered a part of nsile member. The arrangement of the planks and the lengths to be must be determined for each case. In the one shown there is but one in the middle panels where there is the greatest tension. The distance

XY is 12 ft, which is about as small as will serve for the transfer of the te
from *A′* to *B*. In this beam the two outer planks, *A* and *A′*, must be
enough to resist the whole tensile stress in the middle panels because o
joints in *B* and *C*. At the inner end of the second panel there is 58 o
tension which must be carried to the end of the first panel. Because o

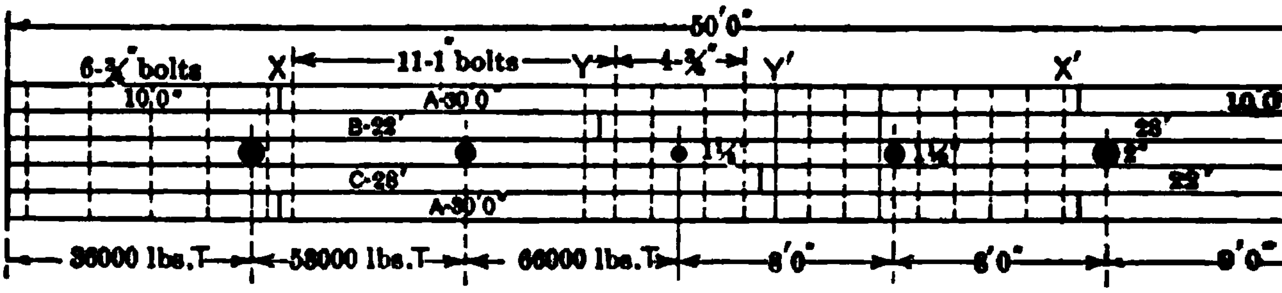


Fig. 25. Plan of Built-up Tie-beam

joints in *A* and *A′* this must be transmitted to *B* and *C* in order to pa
point *X*.

Assuming that 29 000 lb, one-half the tension, is carried on plank *A*
transmitted to *C* by the shear and bearing on the bolts, and dividing t
7 850 lb, the allowable shear on a 1-in bolt, four bolts are found to be nece
But the bearing value of a 1-in bolt in Douglas fir 2 in thick, is only 2 4
which makes twelve bolts necessary. These are required in the distanc
12 ft.

From the distances in Table VII, it is found that the end-bolts must be
from the ends of the planks, say 6 in; this leaves 11 ft, in which distanc
bolts are to be arranged. If four bolts are placed in pairs, two at each e

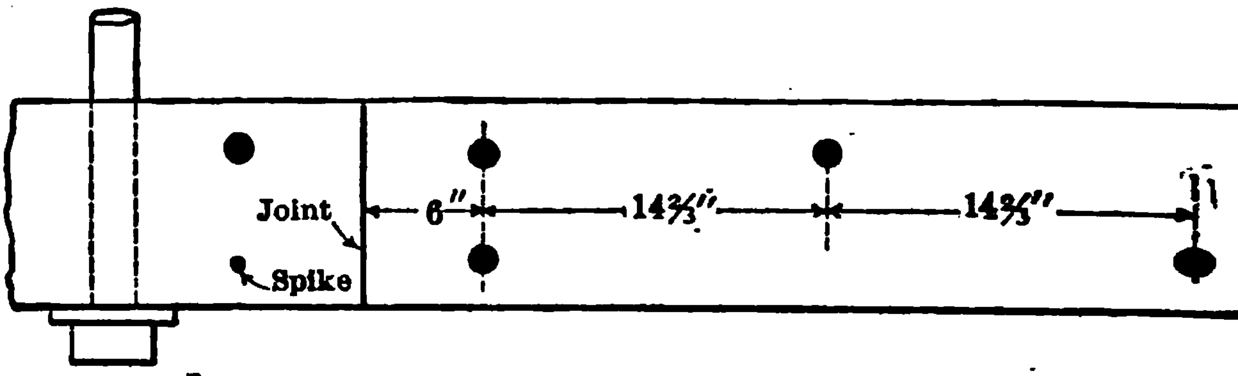


Fig. 26. Elevation of Beam Opposite *X* of Fig. 25

shown in Fig. 26, the intermediate spaces are 14⅔ in. The bolts bind the
together better if they are staggered, as indicated in Fig. 26, and not pla
the middle line.

The number of bolts mentioned is sufficient to make the splice, but
should be bolts in the distance *YY′*, and between the ends and *X* and
bind the planks together. These need not be as large or as close toget
the others; ¾-in bolts spaced 2 ft are sufficient. There should be tw
at the end of the beam. Each bolt should be driven through a hole of th
size as the bolt and the nuts should be screwed up tight.

Case II. Bolts in Girders to Support Brackets. The construction
in Figs. 27 and 28 is commonly used in cases in which the requirements
allow the girder to project its full depth below the joists. The BOLTS sh
Fig. 27 must be investigated for BEARING and SHEAR, and those shown in F
for BEARING, SHEAR and BENDING. In either case the SHEARING VALUE of t
in single shear must equal or be greater than the greater of the forces *S* o

The BEARING per inch on the wood of the girder, when *B* is in inches, i

$$(S + S')/B$$

This must be kept within the values given in Table VII for the timber used. For the case shown in Fig. 28 the BENDING MOMENT in pound-inches is

$$M = SL/2 \quad \text{or} \quad M = S'L/2$$

whichever is the larger. B and L are measured in inches and S in pounds.

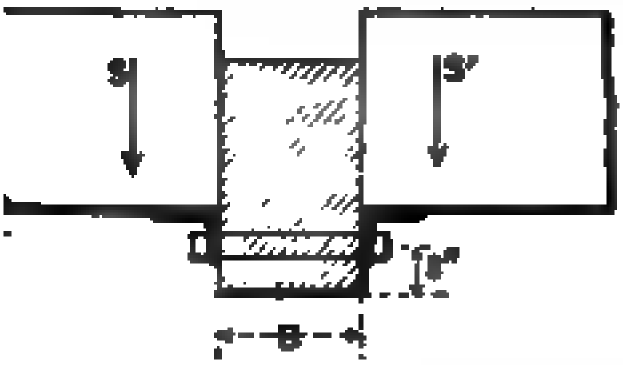


Fig. 27 Fig. 28

Figs. 27 and 28. Bolts Supporting Brackets on Girders

Example 9. For the construction shown in Fig. 27 it is required to determine the number and size of bolts, the Douglas fir girder being 8 by 14 in, with a span of 14 ft, and the Douglas fir joists 3 by 12 in, with a span of 20 ft, center to center of girders. The floor-load, including the floor, is 60 lb per sq ft. The strips are 4 by 3½ by ¾ in.

Solution. The floor-area supported by the girder is 14 by 20 ft. At 60 lb per sq ft, the load is 14 × 20 × 60 = 16 800 lb. The load S, on one side, is 8 400 lb.

A ¾-in bolt has a shearing-value of 4 420 lb. Hence two bolts are necessary to satisfy the shearing condition. The bearing value of the bolt in the wood, across the grain, is, from Table VIII, 187 lb per inch of length, or 1 496 lb for the width of the girder. The number of bolts required, then, is 16 800 divided by 1 496 or approximately 11, which gives a spacing of about 15 in.

Example 10. In the construction shown in Fig. 28, the girder is 6 by 14 in, of Douglas fir and has a span of 12 ft. The joists are 2 by 12 in and have an 18-ft span, center to center of girders. The floor-load is 65 lb per sq ft. There are 3 by 4-in strips on the sides of the girder. The distance L is 3 in. It is required to find the number and size of bolts to be used.

Solution. The total load on the girder is

$$12 \times 18 \times 65 = 14\,040 \text{ lb}$$

$$S = 7\,020 \text{ lb}$$

The bearing load per inch of thickness of the girder is

$$\frac{14\,040}{6} = 2\,340 \text{ lb}$$

The bending moment on one side of the girder is

$$\frac{7\,020 \times 3}{2} = 10\,530 \text{ in-lb}$$

as the force S acts at the center of pressure on the bracket-strip, 1½ in from the edge of the girder.

The shear is 7 020 lb, which requires two ¾-in steel bolts at 4 420 lb for one bolt in Table IX.

The bearing (Table VIII) on a ¾-in bolt is 187 lb per inch of length; therefore it requires thirteen bolts for bearing.

The allowable bending moment on a ¾-in steel bolt is 830 in-lb, from Table
To take care of the 10 530 in-lb requires thirteen bolts. A ⅞-in steel bolt
an allowable bending moment of 1 310 lb-in, making eight of them suffici
The 3 by 4-in pieces may be held in place by thirteen ¾-in bolts spaced 1
on centers, if two of them are placed 6 in from the ends.

Case III. Bolts as Pins in the Joints of Trusses. For TIES STRUTS joined by BOLTS in manner indicated in Figs. 30 and 31 and having thickness B exceeding . the diameter of the bol the number of bolts mus computed for SHEARING, BE ING and FLEXURE.

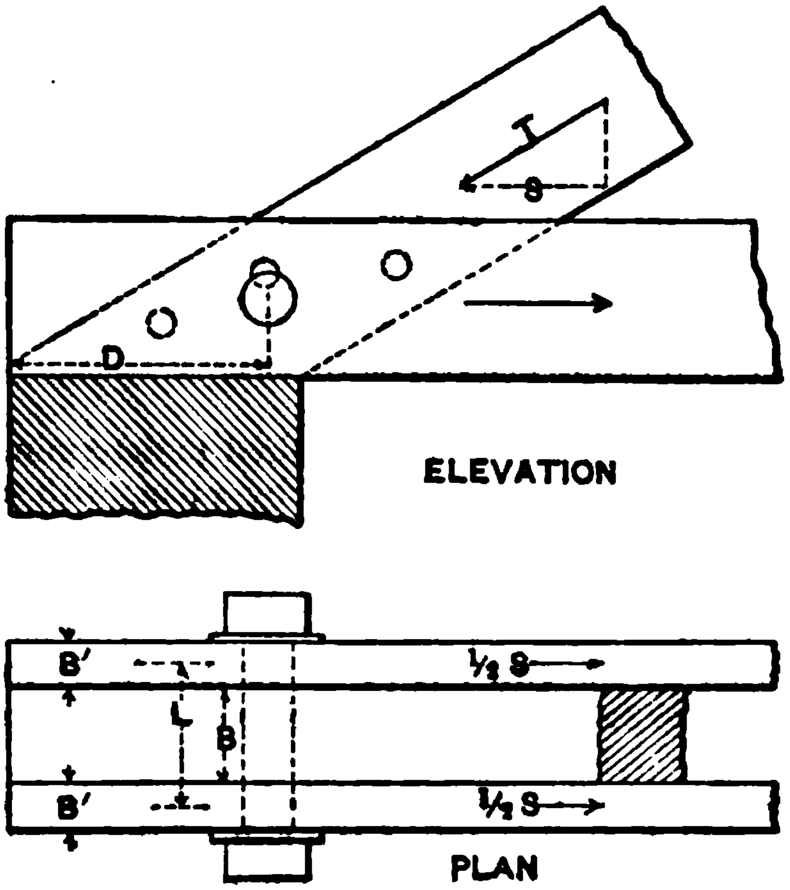


Fig. 29. Bolt through Rafter and Tie-beam

For any of these joints forces are as follows:

The single shear = $S/$

On the sections betwee and B' (Fig. 30)

The bearing on the pi inch of length = S/B or

The greater is to be used.

The bending moment = S

on the assumption of a CON UOUS BEAM, uniformly lo

If there are more bolts one, the quantities obt by the above formulas are to be divided by the number of bolts to find the to be taken care of by one bolt.

In Fig. 29, S is the horizontal component of the thrust T.

Example 11. It is required to determine the diameter of a bolt for a like that shown in Fig. 29. The rafter is 6 by 10 in, of Douglas fir, the tie-beams 3 by 10 in, of the same material, the thrust in the rafter 30 000 lb, and its inclination 30°.

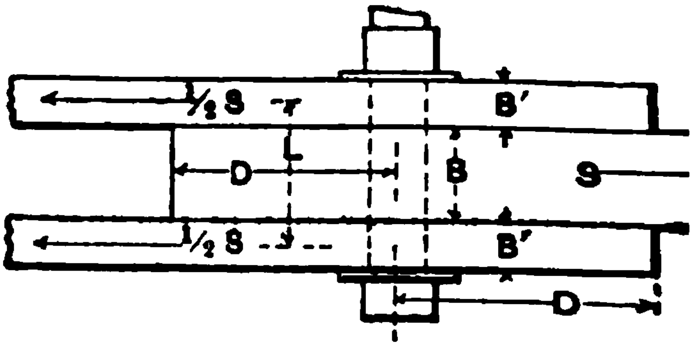


Fig. 30. Bolt in Wooden Tie-beam

Solution. The horizontal component of 30 000 lb at 30° is practically 26 000 lb. Then S = 26 000 lb and the shear = 13 000 lb. B = 6 in and L = 9 in.

Bearing per inch of length on the bolt = 26 000/6 = 4 333 lb
Bending moment = 26 000 × 9/12 = 19 500 in-lb

In Table IX, a 1⅝-in steel bolt is found to be necessary to resist a sh 13 000 lb, and a 2¼-in bolt for a bending moment of 19 500 in-lb. To 4 333 lb end-bearing pressure on 1 in a larger bolt is required than is gi Table VII. Dividing 4 333 by 1 200, the allowable bearing on Dougla 3⅝-in bolt is found to be necessary. This is larger than it is desirable so the joint must be redesigned with a view to reduce the bearing press

bolt. If an 8 by 8-in strut and 4 by 8-in tie-beams are used, B becomes 8 and L 12 in. This gives

Bearing pressure = 26 000/8 = 3 250 lb per inch of length of the bolt
Bending moment = 26 000 × 12/12 = 26 000 in-lb

e total shear at the section on one side of the strut is the same as before. From Table VII it is found that a 2¾-in bolt is large enough to provide for e bearing and that a 2½-in bolt is sufficient for the bending as given in Table . Hence if an 8 by 8-in strut is used, there must be a 2¾-in bolt and e distance D must be 15½ in (Table I).

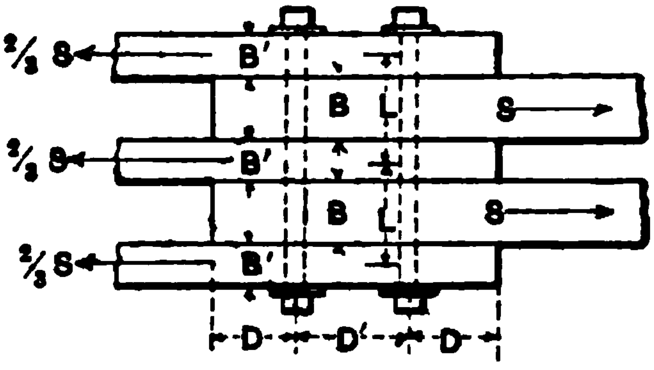


Fig. 31. Bolts in Wooden Tie-beam

Example 12. For the same construction as in Fig. 29 and the same conditions as in the first part of Example , it is required to determine the size the bolts when it is necessary to e three.

Solution. The shear, bearing, and nding moment are the same as in ample 11, but because there are ee bolts each quantity is divided by 3 to determine the force resisted by ch.

Shear = 13 000/3 = 4 333 lb and requires a ¾-in steel bolt (Table IX)
Bearing = 4 333/3 = 1 444 lb and requires a 1¼-in bolt (Table VII)
Bending moment = 19 500/3 = 6 500 in-lb, and requires a 1½-in steel bolt able IX).

In this case the bending moment determines the size of the bolts, which may arranged as shown by the dotted circles in Fig. 29.

Example 13. It is required to determine the diameter of the bolt for the struction shown in Fig. 30, in which the inner beam is of Douglas fir and y 8 in in section, and the outer beams 3 by 8 in, the tension being 24 000 lb.

Solution. $S = 24\,000$; $B = 6$ in; $L = 9$ in.

Single shear on the bolt = 24 000/2 = 12 000 lb
Bearing-pressure per inch of length of bolt = 24 000/6 = 4 000 lb
Bending moment = 24 000 × 9/12 = 18 000 in-lb

From Table IX a 1¼-in steel bolt is found sufficient to resist the shear, and ¼-in bolt large enough to resist the bending. In Table VII the largest bolt sidered, 3 in, is too small in bearing value. Dividing the load to be resisted 1 200 gives 3⅓ in, as the diameter necessary to resist the bearing. The ance D must be 4 000/(2 × 130) + 3⅓ in or 18¾ in.

Example 14. If two bolts are used, one behind the other, it is required to rmine the diameter of the bolt that should be used, the conditions and ing being the same as in Example 13.

Solution. Dividing the quantities obtained in Example 13 by 2,

Single shear = 6 000 lb and requires a ⅞-in steel bolt
Bearing = 2 000 lb and requires a 2-in bolt
Bending moment = 9 000 in-lb and requires a 1¾-in steel bolt

e allowable bearing on a 1¾-in bolt is (2½ %) less than the required amount, ut in general, since the other requirements are more than satisfied, the r bolt would be used. For the 1¾-in bolt, the distance D is 9¼ in. The e between the bolts may be increased somewhat beyond the value given in

Table VII, and they may be located out of the same line as a further pr tion against splitting.

Case IV. Bolt-and-Strap Joints in Trusses. The construction sho Fig. 32 is sometimes used to connect the foot of the rafter of a wooden to the tie-beam. When the distance D is sufficient to resist the shear d the thrust of the rafter, the strap is of value only in holding the rafter in and there are no greater pressures brought upon the BOLT. When it is i sible to make D the necessary length, the BOLT and STRAP must be des to resist the full force in the direction of the STRAP.

As the STRAP is usually not more than from ½ to ¾ in thick, its w is such that the bearing between it and the rafter is small comp with that between the BOLT rafter. The forces acting on the are the only ones that need sideration. These are:

SINGLE SHEAR = $S/2$ = the sion in the strap on one side

BEARING PRESSURE per inc length = S/B, where B is the w of the tie-beam in inches

BEARING PRESSURE per inc length between strap and bolt =

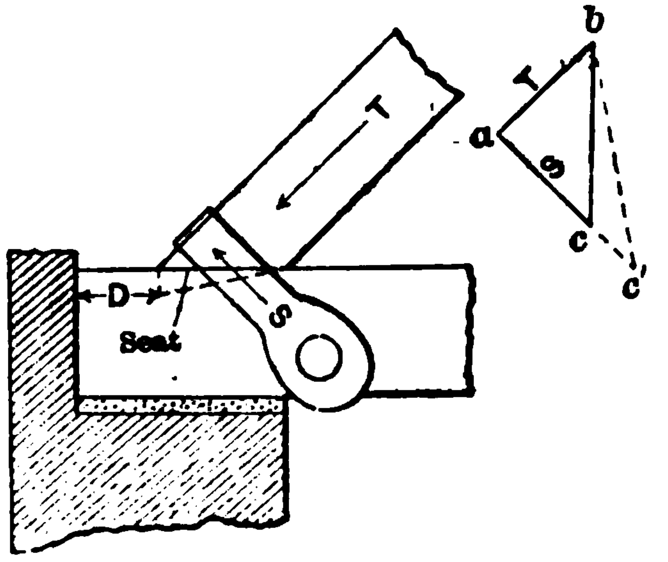


Fig. 32. Strap and Bolt at Foot of Rafter

To find the value of S, the f polygon is drawn as shown at the in Fig. 32. T is drawn parallel t rafter and with a length, to a venient scale, equal to the thrust. From the end a an indefinite line is d parallel to the axis of the strap, and from b another line perpendicular to SEAT of the rafter. These intersect at c, so that ac, measured by the scale used in laying off T, is the magnitude of the force S in the strap. If rafter rests on top of the beam, bc is vertical, but if the tie-beam is dappe shown by the dotted line, the line from b is drawn perpendicular to the bo of the notch, making the intersection at c'. It is seen that notching the beam in this way increases the stress in the strap.

Example 15. It is required to determine the size of a strap and pin to hold the rafter without notching into the tie-beam of a long-leaf yellow- king-post truss. The rafter is 6 by 6 in, is inclined at an angle of 45° a under a compressive stress of 18 000 lb. The tie-beam is 6 by 8 in in sectio

Solution. Since the inclination is 45°, a consideration of the force-pol in Fig. 32 shows ab equal to ac, so that

The force S = the thrust T = 18 000 lb
Single shear on bolt = 18 000/2 = 9 000 lb
Tension in strap on one side = 9 000 lb
Bearing pressure per inch of bolt against wood = 18 000/6 = 3 000
Bearing pressure in pounds per inch between strap and bolt = 9 00
in which t equals the thickness of the strap.

The allowable pressure between the strap and the top of the rafter is 3 per sq in (Table VII), which, on the 6-in rafter, gives

Allowable load per inch of width of strap = 6 × 350 = 2 100 lb

The strap then must be 18 000/2 100 or 8.6 in wide. At 10 000 per in tension the necessary section of the strap is 0.9 sq in, requiring a thickn

at 0.1 in, a sufficient thickness if the strap were strong enough to develop a form pressure over the rafter. It is not good practice, however, to use such material, because of the danger of loss of strength due to corrosion. No al less than ⅜ in thick should be used in such places.

The bearing-pressure per inch, between the strap and the bolt, for a ⅜-in p = 9 000/⅜ = 24 000 lb

The bolt, then, must take a single shear of 9 000 lb, a bearing pressure of lb against the wood for each inch of length, and a bearing of 24 000 lb inch of length against the strap. From Table IX a 1⅛-in steel bolt is cient to resist the shear, from Table V a 2-in bolt is large enough to resist bearing from the strap, and from Table VII a 2¼-in bolt is found necessary esist the 3 000-lb bearing from the wood per inch of length of bolt. This es the 2¼-in bolt satisfactory for the joint.

The pressure from the bolt to the wood, however, is not parallel with the n but inclined at 45°. The allowable pressure against wood across the n is about one-fourth of that with the grain. According to the formula n in Chapter XXVIII, page 1138, the allowable pressure per square inch for case is 612 lb instead of the 1 400 per sq in allowed for direct compression h the grain. The reduced allowable pressure makes it necessary to use a n bolt, say a 5-in bolt, which would be impracticable, for it would almost cut tie-beam in two. It thus appears that this form of joint is not good design a truss of this span. For shorter s the joint may be made in rdance with the requirements n. It has the advantage of not ating any projections below the beam.

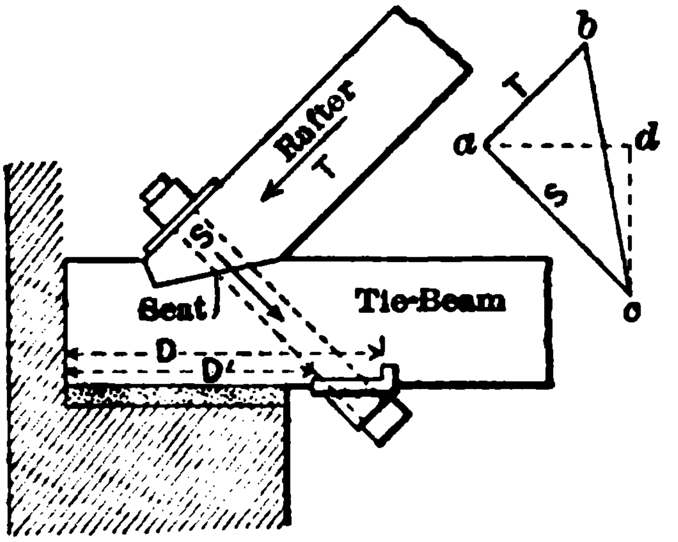
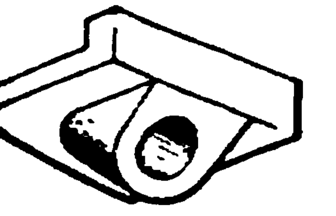


Fig. 33. Bolt in Tension at Foot of Rafter

Case V. Bolts in Tension to ld the Foot of a Rafter. In the t shown in Fig. 33 the bolt is ject to DIRECT TENSION only. e amount of the tension S is found the construction explained in e IV. The rafter may be let into tie-beam or rest on top of it, the ion in the bolt being less in latter case; but it is easier to erect the truss if the rafter is NOTCHED INTO BEAM from 1¼ to 1½ in for ordinary spans and loads, to hold it while the pieces are fitted. After this is done, the holes may be bored exactly where required.


. 34. Special Washer

Whenever S exceeds about 10 000 lb for trusses made of timber for which the highest bearing stresses are allowed, a CAST PLATE, as shown in Fig. 34 and made to fit the inclination of the bolt, should be let into the tie-beam at the head of the bolt to distribute the pressure. The diameter of the hole for the bolt ld be ⅛ in larger than the diameter of the bolt. The distance D must be e sufficient to provide for the horizontal component of S, at the allowed ing stress of the material for shear with the grain.

The horizontal component is found by drawing a vertical line from c and a ontal line from a and measuring ad to the scale of the diagram. For y, this force must be less than the product of the distance D, the width of beam and the allowed shearing-stress given in Table I, page 412.

Example 16. For the same conditions as in Example 15, for the size of members and the thrust in the rafter, it is required to determine the dia of the bolt and the distance *D* for a joint of the type shown in Fig. 33.

Solution. To find *S*, draw *T* equal to 18 000 lb, at a convenient scale, parallel to the rafter. At *a*, draw an indefinite line perpendicular to the r and at *b* a line perpendicular to THE SEAT of the rafter. This makes *S* gr than in Example 15, as *ac* now scales 27 000 lb. From Table IX, a 1¾-in bolt is sufficient to take this in direct tension. The horizontal compo found as directed above, scales 19 000 lb. The width of the tie-beam is which at the allowed shearing-stress, 150 lb per sq in, gives 900 lb as the that must be cared for by each inch of *D*. 19 000 lb divided by 900 gives the required distance *D*. (See, also, Chapter XXVIII, Joints in W Trusses.)

The compression against the grain on the end of the cast-iron washer also be investigated. 19 000 lb divided by the width, 6 in, gives 3 166 lb must be resisted per inch of width of beam. At 1 400 lb per sq in, as an able working stress, this makes it necessary to set the casting 2¼ in int lower side of the beam, which exceeds the depth usual in ordinary pra Some tests made at the Massachusetts Institute of Technology on large tr and reported in 1897, indicated that for a TEST CARRIED TO RUPTURE the st prescribed for usual designs might safely be more than doubled. Tes timber under LONG-CONTINUED LOADING indicate that rupture finally for stresses approximating one-half of those developed in TESTS CARRIE IMMEDIATE FAILURE. This, and the fact that decay may affect the streng the members, emphasizes the wisdom of using CONSERVATIVE WORKING-STR in this material.

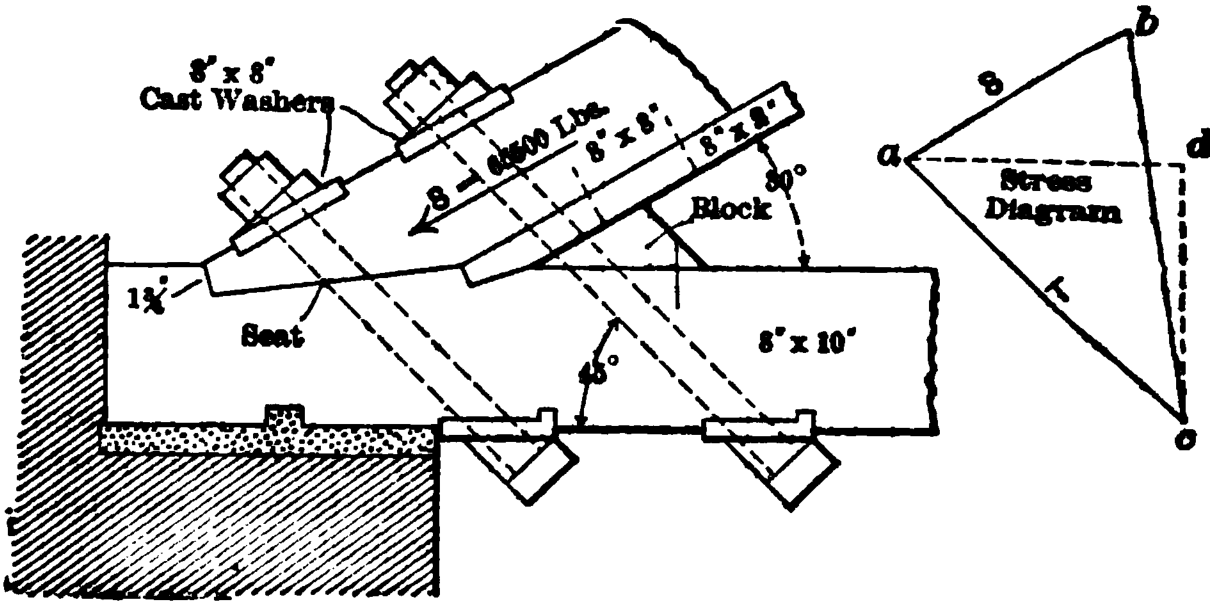


Fig. 35. Joint with Two Bolts in Direct Tension

Example 17. It is required to determine the size of bolts for the joint s in Fig. 35, the thrust being 65 500 lb and the truss-members being ma long-leaf yellow pine.

Solution. The tension in the bolts is found first by drawing the force-po as shown at the right in the figure. To the same scale that *ab* represents 6 *ac* represents 96 500 lb. If the load is equally divided between the bolts, has a tension of 48 250 lb. From Table IX this force requires a 2¼-in bolt.

The horizontal component *ad* is 68 350 lb, which must be resisted by the ing strength of the wood between the end of the cast-iron washer on the

of the tie-beam and the end of the beam resting on the wall. At 150 lb sq in, this requires 68 350/150, or 455 sq in. If the beam is 8 in wide, this ires a length of 57 in along the beam from the washer to the end.

he bearing of the cast-iron washer against the end-fibers of the tie-beam is 68 250 lb. At an allowable pressure of 1 400 lb per sq in the depth of washer should be 68 350/(8 × 1 400) = 6.1 in. This would almost cut the m in two. The ultimate strength of the wood in compression is about five s the working stress, and since a considerable part of the horizontal force be resisted by the body of the bolt as well as by the friction of the washer, probable that with washers ¾ in thick there would be little sign of weak- at the joint even when the truss is fully loaded.

heoretically the washers on the top surface of the rafter should be deter- ed by the allowable working stress in compression across the fibers. This ong-leaf pine is taken at 350 lb per sq in (Table VI, page 454). The area, , is 48 250/350, or 138 sq in. This requires a washer 11¾ in square. e 8 by 8-in washer used, assumes a pressure of 755 lb per sq in, but as the tests he Forest Service of the United States Department of Agriculture give 3 480 per sq in as the elastic limit for long-leaf yellow pine, it is very likely that re would be no signs of injury at this point, other than a SLIGHT INDENTA- , when the truss is fully loaded.

CHAPTER XIII

BEARING-PLATES AND BASES FOR COLUMNS, BEA AND GIRDERS. BRACKETS ON CAST-IRON COLUMNS*

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1. Bearing-Plates and Bases

The Purpose of Bearing-Plates or Bases. When a heavily loaded umn, beam or girder is supported on a masonry wall or pier, a BEARING-PLAT BASE of suitable dimensions must be used to distribute the load so that pressure will not exceed the safe BEARING STRENGTH of the masonry (Tabl

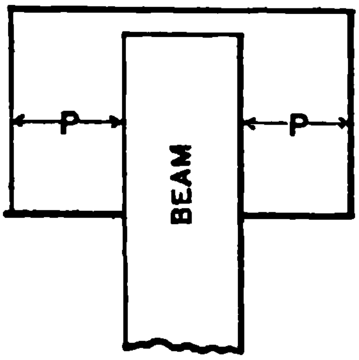


Fig. 1. Simple Bearing-plate

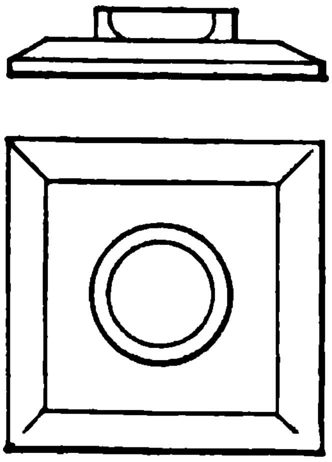


Fig. 2. Beveled Cast-iron Plate with Pin

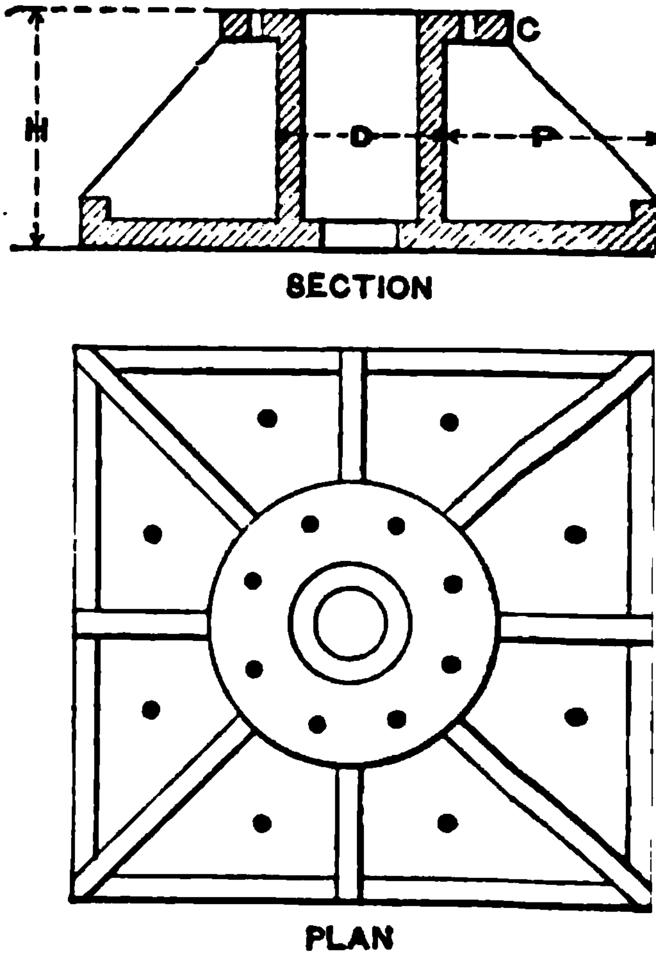


Fig. 3. Ribbed Cast-iron Plate

The BEARING-PLATE is designed to be stiff enough to distribute the pre under it uniformly, and its area is determined by dividing the load on the allowable pressure per unit of area (Table II).

Simple Bearing-Plates. Fig. 1 shows a SIMPLE BEARING-PLATE und beam. It may be a steel or cast-iron rectangular plate of sufficient thic to prevent its bending at the edge of the beam from the pressure of

* See, also, Chapter XIV, Subdivisions 8 to 11.

...nry below. For anchors for steel beams on bearing-plates, see Chapter ..., page 619.

...ast-Iron Plates with Pin. Fig. 2 is a cast-iron PLATE WITH A DOWEL-PIN ...t inside the shell of a cast-iron column, or into a recess cut in the bottom ... wooden one. The pin holds the base-plate in position.

...ast-Iron Ribbed Bases. Fig. 3 is a cast-iron RIBBED BASE for a large ...ndrical cast-iron column, capable of supporting a load heavy enough to ...k a plate similar to the one shown in Fig. 2, at the edges of the column, ...ss the plate were made unduly thick.

Table I. Allowable Bearing Pressure on Different Kinds of Masonry

Kind of masonry	Allowable pressures, Lb per sq in	Allowable pressures, Tons per sq ft
From the building laws of New York, 1917		
...ick, in lime mortar	110	8
in lime-and-cement mortar	160	11½
in Portland-cement mortar	250	18
...bble masonry, in Portland-cement mortar	140	10
...ncrete, Portland cement, 1 : 2 : 4	500	36
From the building laws of Chicago, 1916		
...ubble, in lime mortar	60	4.32
in Portland-cement mortar	100	7.2
...ared rubble, in lime mortar	120	8.6
in Portland-cement mortar	200	14.4
...hlar, limestone, in Portland-cement mortar	400	28.8
granite, in Portland-cement mortar	600	43.2
...ncrete, Portland-cement, 1 : 2 : 4, hand-mixed	350	25.4
machine-mixed	400	28.8

...he Bases of the Steel Cores of Composite Columns used in reinforced-...rete construction have areas sufficient to distribute the loads of the columns ... the concrete in the footings at the allowable working stress of the concrete. (..., also, page 474, Figs. 14 and 15.)

...ample 1. The basement-columns of a warehouse are designed for a load ...12 000 lb each. It is required to determine the size of the base-plates to ...on the concrete foundations. (Table II used.)

...lution. At an allowable pressure of 208 lb per sq in, the required area is ...000/208 or 1 020 sq in, or about 32½ in square. The plan and section of ...ase-plate is shown in Fig. 3.

...ms of Base-Plates. For small columns and wooden posts with light ..., PLAIN FLAT PLATES of cast iron or steel are generally used. The cast-iron ...may have a raised ring or cross to fit inside a hollow metal column, or ...ed, from 1½ to 2 in in height for a wooden one. If the plate is very thick ...er edges may be beveled to save weight, as shown in Fig. 2, but no part should be less than about ¾ in thick.

Table II. Allowable Loads on Standard, Steel Bearing-Plates on Wal

Bearing on wall, in	Size of plate, in	Safe bearing value of plate in pounds: Bricks laid in mortar of: Lime, 112* lb per sq in	Lime and cement, 162* lb per sq in	Cement, 2 lb per sq
6	6×6	4 070	5 800	7 500
	6×8	5 400	7 800	10 000
	6×10	6 700	9 700	12 500
8	8×8	7 200	10 200	13 300
	8×10	9 000	12 100	16 600
	8×12	10 700	15 500	20 000
10	10×10	11 200	16 200	20 800
	10×12	13 450	19 500	25 200
	10×14	15 700	22 700	27 900
12	12×12	16 150	23 300	30 000
	12×14	18 800	27 400	35 000
	12×16	22 000	31 200	40 100
	12×18	24 200	34 500	45 000
14	14×14	22 100	31 800	40 800
	14×16	25 000	36 300	46 600
	14×18	28 200	40 800	52 400
	14×20	31 400	45 400	58 200
16	16×16	28 700	41 500	53 200
	16×18	32 300	46 600	59 800
	16×20	35 800	51 900	66 700
	16×22	39 500	57 000	73 200

* These values are slightly different from those of the New York Code (1916)

Ribbed Bases. If the calculated size of a bearing-plate is so large th projection beyond the edge of the column would be more than about 6 RIBBED BASE similar to that shown (Fig. 3) for a cylindrical column is For such bases it is unnecessary to consider the transverse stresses. these bases are bolted to the columns they add greatly to the general sta of the supporting members because of the greater width of such bases.

Proportions of Ribbed Bases. The HEIGHT H of this type of base sl be approximately equal to the PROJECTION P, and the DIAMETER D equ the diameter of the column. The projection C should be at least 3 in to p the bolting of the column to the base. The THICKNESS of all parts of the c should be the same and approximately equal to the thickness of the co shell. There must be no thin webs as they result in breakage from shrin stresses.

Base-Plates for Steel Columns are usually made of STEEL PLATES and s as shown on the channel-columns in Chapter XIV, Figs. 17, 18 and 19. iron bases are sometimes used for very heavy columns. If conditio favorable to the action of corrosion the cast iron is to be preferred.

The Area of Bearing-Plates under Beams and Girders is found in the ne manner as the area of plates under columns. If the load on the beam is iformly distributed over the beam or concentrated at its middle, the required a of the plate is one-half the total load on the beam divided by the allowable aring per unit of area on the masonry; but if the load is a moving load, the eatest possible end-reaction must be divided by the allowable bearing. For ample, a heavily loaded truck standing near the end of the beam causes a essure on the bearing-plate much greater than one-half its weight. The e reaction for the actual conditions must be found by the methods explained Chapter IX.

The Thickness of the Bearing-Plate is found by the formula used to ermine the flexure of beams. It must be determined in each case. For a pical case the forces acting are shown in Fig. 5, which represents a transverse

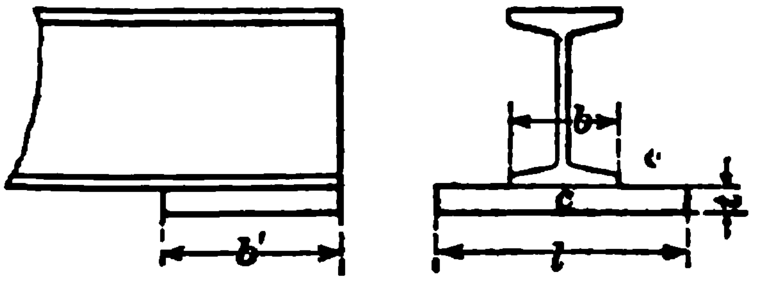


Fig. 4. Simple Bearing-plate under I Beam

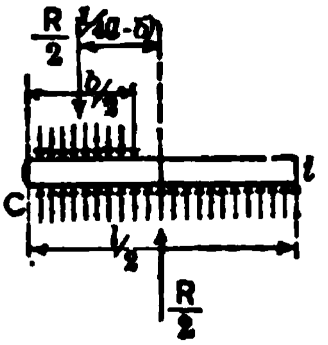


Fig. 5. Forces Acting on Half of Bearing-plate

tical section through one-half the plate. The vertical section at C, and ough and parallel with the web of the I beam, is taken through the center of plate, which is the dangerous section, or section of maximum bending mo nt.

In Figs. 4 and 5, b' is the bearing depth on the wall;
l is the length of the plate, parallel with the wall;
b is the width of the flange of the beam;
R is the load on the bearing-plate.

eplacing the uniform loads by the equivalent forces at the center of gravity each, these forces are represented by the longer arrows. The bending mo t at the section at c is the same as the moment of the concentrated forces, ing

$$M = (R/2 \times l/4) - (R/2 \times b/4)$$
$$M = R/2 \times (l - b)/4$$

s is equal to the resisting moment at the same section c, or, at stress S, c, in which I/c is the section-factor. (See Chapter XV.) This reduces to $^2/6$. Equating the bending moment and the resisting moment there results

$$St^2b'/6 = R(l - b)/8$$

$$t = 0.866\sqrt{R(l - b)/Sb'}$$

r $S = 3\,000$ for cast iron, this reduces to

$$t = 0.0158\sqrt{R(l - b)/b'} \qquad (1)$$

r $S = 16\,000$ for steel plates, it becomes

$$t = 0.00685\sqrt{R(l - b)/b'} \qquad (2)$$

Example 2. It is required to determine the length and thickness of a cas iron bearing-plate under a wooden beam which is 10 in wide and supports load of 24 000 lb. The plate is 8 in wide and bears that width on a brick w laid up in lime mortar.

Solution. The load on the plate is $24\,000/2 = 12\,000$ lb. From Table I the area of the plate is $12\,000/112 = 108$ sq in. Hence, if the width of t plate is 8 in, its length must be $13\frac{1}{2}$ in. Then, from Formula (1)

$$t = 0.0158\sqrt{12\,000\,(13\tfrac{1}{2} - 10)/8} = 1.15 \text{ in}$$

A plate $1\frac{1}{4}$ in thick would be used.

Example 3. It is required to determine the length and thickness of a st bearing-plate under the end of a 24-in 79.9-lb I beam supported on a 12-in br wall laid up in lime-and-cement mortar and carrying a load of 60 000 lb. T width of the flange of the beam is 7 in. (See Table I.)

Solution. The load on the plate is $60\,000/2 = 30\,000$ lb
The area of the plate $= 30\,000/160 = 187\frac{1}{2}$ sq in
The length of the plate is $187.5/12 = 15.6$ in

Then, from Formula (2)

$$t = 0.00685\sqrt{30\,000\,(15.5 - 7)/12} = 1 \text{ in}$$

Standard Sizes of Steel, Wall Bearing-Plates. These are given in Ta II, and are based upon ALLOWABLE PRESSURES of 112, 162 and 208 lb per sq These UNIT PRESSURES are based upon the ALLOWABLE PRESSURES of the N York and Philadelphia building laws which are pressed in tons per square foot. Because of complicated formula on which the thickness depe it is best to compute the thickness for each case.

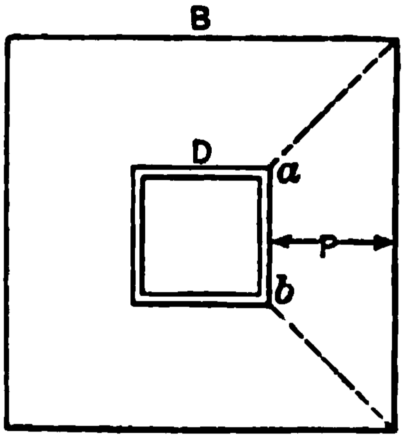


Fig. 6. Flat Bearing-plate for Column

Bearing-Plates under Columns. The gen rules already given for the proportions of RIB BASES similar to that shown in Fig. 3 are a suffic guide for detailing such bases; but in case sin FLAT PLATES are used under columns, their thick must be computed according to the principles gov ing bending. The stress in a FLAT PLATE suppo at the middle and subjected to a uniform load ca be determined by the ordinary methods of mecha The approximate solution here given is gene used in the design of BASE-PLATES and COLUMN-FOOTINGS. It gives va found to be safe in practice.

In Fig. 6, let B = the length of the side of the plate as determined by allowable pressure on the supporting masonry;
D = the side or diameter of the column;
$P = (B - D)/2$ = the projection of the plate;
t = the thickness of the plate;
A' = the area of the plate outside the column;
w = the allowable bearing pressure on the masonry due to load on the column.

Then in Fig. 6, the pressure on one-fourth of A', shown enclosed by the d lines in the figure, causes shearing and bending stresses in the section o plate along the line ab. Considering the part enclosed and taking mo about the section ab, the following equation is obtained from the usual

-moment formula. (See Chapter XV, page 557.) That is, the resisting ment equals the bending moment, or

$$SI/c = \tfrac{1}{4}A'Pw$$

r the rectangular section at ab, this may be written

$$St^2D/6 = \tfrac{1}{4}A'Pw$$

ence $$t = \sqrt{3\,A'Pw/2\,SD}$$

ich becomes for $S = 3\,000$

$$t = 0.0224\sqrt{A'Pw/D}$$

d for $$S = 16\,000$$

$$t = 0.0097\sqrt{A'Pw/D}$$

Example 4. It is required to determine the size and thickness of a cast-iron ring-plate to be used under a wooden post 12 in square in cross-section and signed for a load of 115 200 lb. The plate is to be set on brickwork laid in ent mortar in New York. (See Table I.)

Solution. The required area of the base is 115 200/250 = 461 sq in. $\sqrt{461}$ 21.47 and a 22-in. square plate would be used.

Then
$$A' = 461 - 144 = 317 \text{ sq in}$$
$$P = (22 - 12)/2 = 5 \text{ in}$$
$$D = 12 \text{ in}$$
$$w = 250 \text{ lb per sq in}$$

nce $$t = 0.0224\sqrt{317 \times 5 \times 250/12} = 4 \text{ in}$$

This thickness may be beveled to 1½ in at the edge. The computed thickness greater than is usual for such plates, some formulas having more practical stants which really assume a stress of about 10 000 lb per sq in in cast iron bending.

If the plate is made of steel

$$t = 0.0097\sqrt{317 \times 5 \times 250/12} = 1\tfrac{3}{4} \text{ in}$$

2. Bearing-Brackets on Cast-Iron Columns

he Usual Column-Connections for fastening beams and girders to cast-i columns are shown in Fig. 7.* The end of the beam or girder is set on ELF P, under which is a BRACKET-SUPPORT C, cast on the side of the column. a single beam, one bracket is sufficient; for wide beams or girders there ld be two ribs. The ends of the beams are fastened to the column by ing to LUGS L, cast on the column above the bracket. Sometimes a column stened by bolts passing through the bottom flange of the beam and through shelf-plate. This connection greatly decreases the lateral stability of a ing and should not be used.

he Shelf and Brackets, when loaded, are subject to SHEARING and BEND-STRESSES. The SHEAR at the outer surface of the column-shell is equal e end-reaction of the beam it supports. The BENDING-STRESS is due to the cation of the load on the shelf-plate at some distance from the surface of lumn. It causes a tension at the top of the bracket which tends to tear e shell of the column, and causes, also, a compression at the foot of the The THICKNESS OF THE RIB must be great enough to withstand the com-n from the load above; and since the stress is variable along a section,

* See also, Figs. 5 and 7, pages 457 and 458.

as along the line X, a rough approximation may be made by assuming t stress at the extreme edge to be twice the average stress, and by further assu ing that the section in the rib takes care of all the compression. This make unnecessary to find the CENTER OF GRAVITY and the MOMENT OF INERTIA of t section at X, both of which must be known if the FLEXURE-FORMULA is us This procedure, also, makes unnecessary any assumption as to the true positi of the CENTER OF PRESSURE on the top surface of the bracket. With the thi ness of rib given in the tables there is an ample FACTOR OF SAFETY for any lo

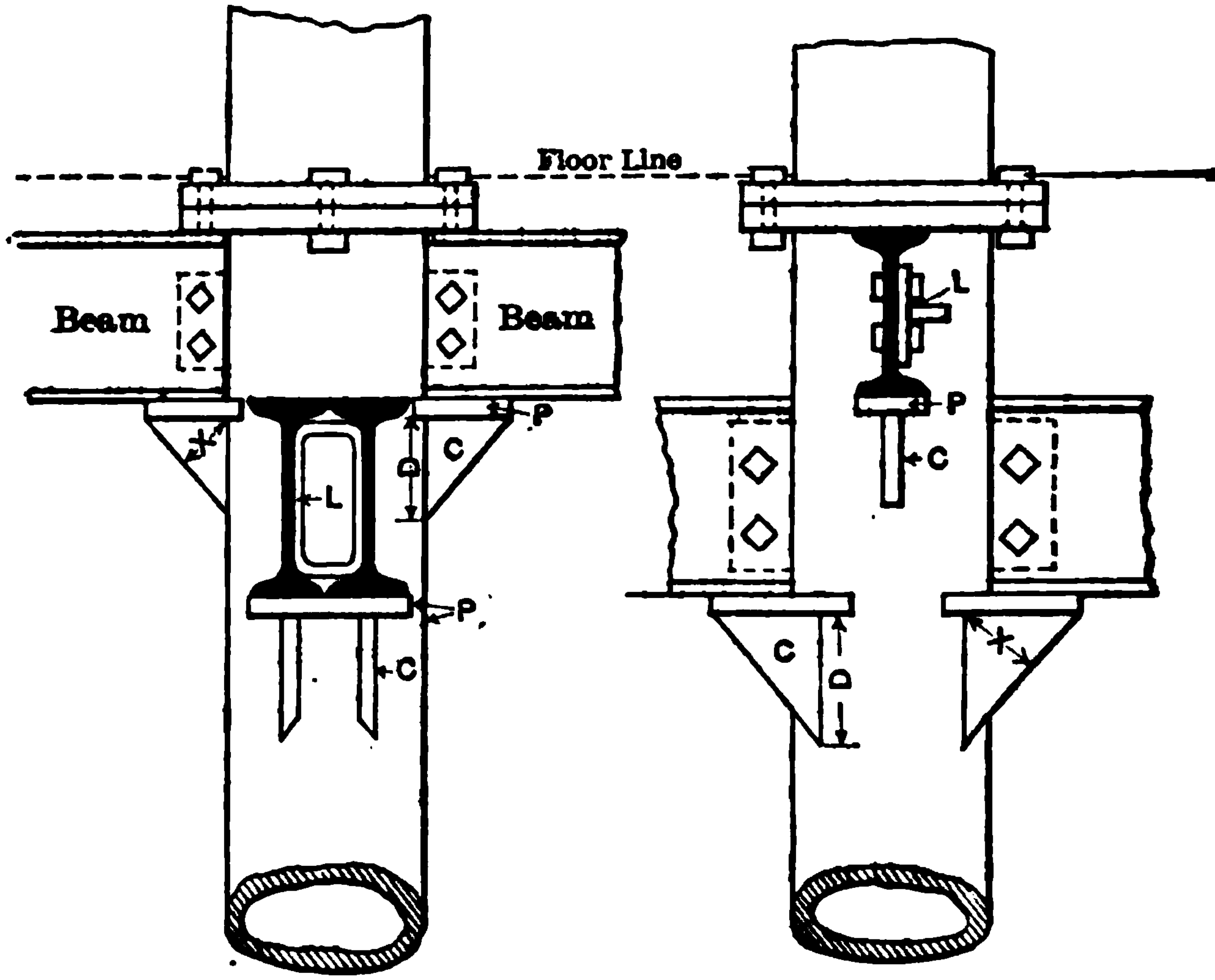


Fig. 7. Cast-iron Columns with Bearing-brackets

that may be applied through a beam. The double ribs are required when beams are used, not for strength, but to prevent the failure of the shelf ECCENTRIC LOADING.

Tests of Cast-Iron Brackets. Brackets of cast-iron columns tested by New York Building Department gave a SHEARING STRENGTH of 4 200 lb sq in on the section at the column when the load was applied at the end of bracket, and an average of 8 000 lb per sq in when the load was distributed the bracket-shelf. The RANGE OF STRESS in the first case was from 2 45 5 600 and in the second from 4 100 to 10 900 lb per sq in. In seventeen ou twenty-two tests the MANNER OF FAILURE was the tearing out of a hole in body of the column. It appears that when the thickness of the rib and is the same as that of the shell of the column, there is generally ample stre for the support of beams and girders; but that in the case of very he loaded beams, the SHEARING and CRUSHING STRENGTH should be investig From the results of the tests mentioned, a low WORKING STRESS FOR S must be assumed.

The Bevel of Brackets. If the shelf P (Fig. 7), on which the beam is cast SQUARE with the column, when the beam deflects, the load is broug the extreme end of the bracket, causing an increased bending-stress in

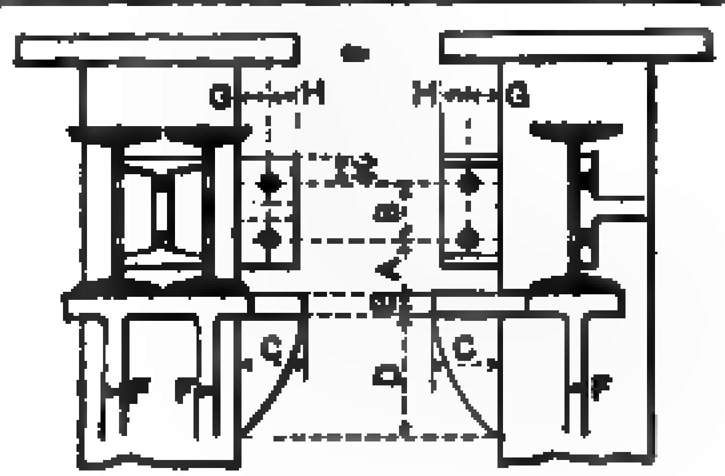
acket and connections and tending to tear a hole in the column-shell. T
oid this the bracket-shelf should be sloped downward, away from the colum
d should have a BEVEL of ⅛ in to the foot.

Standard Connections for Cast-Iron Columns. Table III, publishe
iginally in the Passaic Rolling Mill Handbook, and widely used by othe
anufacturers, will be found useful when detailing cast-iron columns.

Table III. Standard Connections for Cast-Iron Columns

All dimensions are in inches

Depth of beam	A	B	C	D	E	F	G	H	K	Thickness of lugs	Holes cored for ¾-in bolts
20	5	5	6	10½	1½	1½	2	1½	2	1	
18	4	5	6	10½	1½	1½	2	1½	2	1	
15	4	3¾	5½	9½	1½	1¼	2	1½	1¾	1	
12	3	3	4½	7¾	1¼	1¼	2	1½	1½	1	



Depth of beam	A	B	C	D	E	F	G	H	K	Thickness of lugs	Holes cored for ¾-in bolts
10	3¼	3¾	4	7	1¼	1	2	1½	1½	1	
9	3	3	4	7	1	1	2	1½	1½	1	
8	2¾	[illegible]	4	7	1	1	2	1½	1½	¾	
7	2¾	2½	4	7	1	1	2	1½	1¼	¾	

CHAPTER XIV

STRENGTH OF COLUMNS, POSTS AND STRUTS

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1. General Principles and Definitions

Slenderness-Ratio. The manner in which a material fails under compre or pressure depends not only upon its nature, but also upon its dimension form. The ratio of its length, in inches, to its diameter or least lateral di sion, in inches, l/d, or the ratio of its length to its least radius of gyratio is its SLENDERNESS-RATIO.

The following average limits of slenderness-ratios are generally recog in engineering practice:

Solid wooden columns, l/d from about 8 to 30;
Solid wooden columns, l/r from about 30 to 100;
Hollow cast-iron columns, l/d from about 8 to 20;
Hollow cast-iron columns, l/r from about 10 to 70;
Steel columns, l/r from about 30 to 150.

Three Classes of Columns. (1) The actual compressive strength of terial must be determined on very short specimens in which there is no ten to bend or to buckle. (2) The load required to break the specimen do change much until l/d is about 8 or 10, or l/r is about 25 or 30. When ratios are exceeded, the specimens tend to fail partly by direct compressio partly by bending. (3) When l/r is greater than about 200, the colum by bending.

2. Strength of Short Wooden Columns

The Safe Load for a Short Wooden Column, the length of which more than 10 times the least dimension, may be computed by the formula

$$\text{Safe load} = \frac{\text{area of cross-section} \times S}{\text{factor of safety}}$$

in which S denotes the crushing strength of the given material as stat Table I.

The Factor of Safety to be selected depends upon the place where th umn is used, the load which comes upon it, the quality of the material an large measure, upon the value given to S. For lumber of ordinary quality taining no very bad knots, a FACTOR OF SAFETY of five may be used; or, in words, the safe stress per square inch of section-area may be taken as on of the values given in Table I. If the column is badly season-checked, grained, or contains bad knots, a larger factor, say six or seven, should be The character of the load, also, should be taken into consideration in deter the factor of safety. Thus for a wooden post supporting a brick wall a factor should be used than for one supporting a floor, as in the former ca full load is at all times on the post, and the least reduction of its section-

of fire might cause it to give way. Wooden posts supporting machinery, or ...den struts in railway bridges, should have a factor of safety of from six to ...it, if the values of S given in Table I are used.

...ble I.* Average Crushing-Loads in Pounds per Square Inch, for Building Materials

Materials	Crushing-loads, lb per sq in	Materials	Crushing-loads, lb per sq in
	S		S
...r stone, brick, concrete ...and masonry, see Chapter V		Woods (continued)	
		Cypress	3 500
		Hemlock	4 000
Metals		Oak, white	5 000
		Pine, long-leaf yellow	5 000
...ast iron	80 000	Pine, short-leaf yellow	4 000
...rought iron	55 000	Douglas fir	4 500
...teel, rolled shapes	60 000	Pine, Norway	3 500
		Pine, white	4 000
Woods, with the grain†		Redwood, California	4 000
...edar	3 500	Spruce	4 500
...hestnut	4 000	Whitewood	3 000

* See, also, Table XVI, page 647, and Table I, page 1138.

† These are values for wooden columns under 15 diameters in height and are, of ...e, average values. For the safe loads, per sq in, on timbers, perpendicular to ...grain, see Table VI.

...ample 1. What is the safe load for a long-leaf yellow-pine column, 10 by 10 ...n cross-section and 12 ft long, using a factor of safety of 5?

...ution. Area of cross-section = 100 sq in; safe load per sq in = 5 000/5 = 1000; ...0 × 100 = 100 000 lb.

...ample 2. It is required to support a brick wall weighing 80 000 lb by a ...uglas-fir column 11 ft long. What should be the cross-section of the column?

...lution. As previously stated, for these conditions it would be wise to use a ...or of safety of 6. Then the safe resistance per square inch of section-area = ...0/6 = 750; 80 000/750 = 106 sq in required, about equivalent to a 10 by ...n cross-section.

. Strength of Wooden Columns or Struts Over Ten Diameters in Length. Formulas

...ormulas for Wooden Columns. If the length of a solid column exceeds ...t ten times its least cross-dimension it is liable to bend under the load, and ...ce to break under a less load than would break it if it were shorter and of ...same cross-section. To deduce a formula which will make the proper allowance for the length of a column has been the aim of many engineers, but their ...ulas have not always been exactly verified by actual results.

...ntil recently the formulas of Lewis Gordon and C. Shaler Smith have been ... generally by engineers, but the extensive series of tests made by the Govern...t at Watertown, Mass., on full-sized wooden columns, showed that these ...ulas did not agree with the results there obtained. James H. Stanwood ...e year 1891 plotted the values of all the tests made at the Watertown ...enal up to date on full-size wooden columns. From the results thus obtained

he deduced the following STRAIGHT-LINE FORMULA for long-leaf yellow-pine ε white-oak columns:

$$\text{Safe load per square inch} = 1\,000 - 10 \times \frac{\text{length in inches}}{\text{breadth in inches}}$$

The author has carefully compared this formula with the results of act tests, and with other formulas,* and believes that for timber without seri defects and with not more than 10 or 12% of moisture, it meets the actual c ditions as nearly as any other formula. He has therefore prepared Table III, IV and V for the strength of round and square columns of the sizes gener used in practice. Of course other formulas must be used when required by tain city building laws. For other sizes the loads can easily be computed the formulas. For columns having bad knots or other defects, or more t 10 or 12% of moisture, or which are to be exposed to the weather or known be eccentrically loaded, a deduction of from 10 to 25% should be made from values given in the tables.

The loads for columns of other species of wood were computed by the follow formulas of the same form as that of Formula (2):

For Douglas fir and spruce,

$$\text{Safe load per square inch} = 850 - 8.5 \times \frac{\text{length in inches}}{\text{breadth in inches}}$$

For chestnut, hemlock, short-leaf yellow pine and white pine,

$$\text{Safe load per square inch} = 750 - 7.5 \times \frac{\text{length in inches}}{\text{breadth in inches}}$$

For cedar, cypress, redwood, Norway pine and whitewood,

$$\text{Safe load per square inch} = 625 - 6 \times \frac{\text{length in inches}}{\text{breadth in inches}}$$

In these formulas the breadth is the least side of a rectangular column, or diameter of a round column. The round columns were computed for the inch, to allow for being turned out of a square column, of the next size la The formulas were used only for columns with a diameter or least side exc ing 12 diameters for yellow pine and white oak, and exceeding 10 diam for other woods.

4. Tables of Safe Loads for Wooden Columns

Tables II, III, IV and V give the safe loads in pounds for round and sc wooden columns of different cross-sections and lengths and of different kin wood. They were computed from formulas as explained above and ar favorable conditions of material, seasoning and position in buildings.

* There are many formulas for the safe loads per square inch of cross-section of w columns. Among those frequently used are the following:

American Railway Engineering and Maintenance of Way Association,

$$P/A = S\,(1 - l/60\,d)$$

Department of Agriculture,

$$P/A = S\,(700 + 15\,l/d)/(700 + 15\,l/d + l^2/d^2)$$

Winslow Formula (Chicago Law),

$$P/A = S\,(1 - l/80\,d)$$

In these formulas, P is the safe load in pounds, A the area of the cross-section in inches, P/A the safe load in pounds per square inch, S the safe end-bearing comp per square inch, l the length in inches and d the least side or diameter in inches. formulas give smaller safe loads than those of Tables II, III, IV and V; but as th of these tables are to be DECREASED for unfavorable conditions and the loads dete from the three formulas mentioned INCREASED for favorable conditions, the resu about the same.

Table II. Safe Loads in Pounds for Long-Leaf Yellow-Pine and White-Oak Columns, Round and Square

Size of column in inches	Length of column in feet								
	8	10	12	14	15	16	18	20	24
4×6..........	18 200	16 800	15 360	.....	.....	.....	.....	.....	.....
5½ round.....	19 590	18 760	17 550	16 500	.....	.....	.....	.....	.....
6×6..........	30 200	28 800	27 400	25 900	25 200	24 500	.....	.....	.....
6×8..........	40 300	38 400	36 500	34 600	33 600	32 600	.....	.....	.....
6×10..........	50 400	48 000	45 600	43 200	42 000	40 800	.....	.....	.....
7½ round.....	38 540	37 130	35 710	34 300	33 590	32 890	.....	.....	.....
8×8..........	64 000	54 400	52 500	50 600	49 600	48 600	46 700	.....	.....
8×10..........	80 000	68 000	65 600	63 200	62 000	60 800	53 400	.....	.....
8×12..........	96 000	81 600	78 700	76 800	74 400	73 000	70 100	.....	.....
9½ round.....	70 900	61 970	60 190	58 350	57 429	56 580	54 800	.....	.....
10×10..........	100 000	100 000	85 600	83 200	82 000	80 800	78 400	76 000	.....
10×12..........	120 000	120 000	102 700	99 800	98 400	97 000	94 100	91 200	.....
10×14..........	140 000	140 000	119 800	116 500	114 800	113 100	109 800	106 400	.....
11½ round....	103 900	103 900	90 912	88 730	87 690	86 550	84 160	82 290	.....
12×12..........	144 000	144 000	144 000	123 800	122 400	121 000	118 100	115 200	109 440
12×14..........	168 000	168 000	168 000	144 500	142 800	141 100	137 800	134 400	127 680
12×16..........	192 000	192 000	192 000	165 100	163 200	161 300	157 400	153 600	145 920
14×14..........	196 000	196 000	196 000	196 000	170 900	169 100	165 800	162 400	155 800
16×16	256 000	256 000	256 000	256 000	229 100	225 300	221 400	217 600	209 900
18×18..........	324 000	324 000	324 000	324 000	324 000	289 400	285 100	280 800	272 160
20×20..........	400 000	400 000	400 000	400 000	400 000	400 000	356 800	352 000	342 400

Table III. Safe Loads in Pounds for Douglas-Fir and Spruce Columns, Round and Square

Size of column in inches	Length of column in feet								
	8	10	12	14	15	16	18	20	24
×6..........	15 500	14 280	13 050	.....	.....	.....	.....	.....	.....
½ round.....	16 650	15 790	14 900	14 030	.....	.....	.....	.....	.....
×6..........	25 704	24 480	23 256	22 032	21 420	20 808	.....	.....	.....
×8..........	34 272	32 640	31 008	29 376	28 560	27 744	.....	.....	.....
×10..........	42 840	40 800	37 760	36 720	35 700	34 680	.....	.....	.....
½ round.....	32 740	31 540	30 340	29 140	28 540	27 940	26 740	.....	.....
×8..........	47 870	46 240	44 600	42 970	42 160	41 340	39 710	.....	.....
×10..........	59 840	57 800	55 760	53 720	52 700	51 680	49 640	.....	.....
×12..........	71 808	69 360	66 910	64 460	63 240	62 000	59 560	.....	.....
½ round.....	54 150	52 650	51 150	49 580	48 820	48 070	46 570	.....	.....
×10..........	85 000	74 800	72 760	70 720	69 700	68 680	66 640	64 600	.....
×12..........	102 000	89 760	87 300	84 860	83 640	82 400	80 000	77 500	.....
×14..........	119 000	104 700	101 860	99 000	97 580	96 150	93 300	90 400	.....
½ round....	88 290	79 100	77 250	75 400	74 470	73 550	71 700	69 850	66 160
×12..........	122 400	110 160	107 700	105 260	104 040	102 800	100 360	97 920	93 000
×14..........	142 800	128 520	125 660	122 800	121 380	119 950	117 100	114 240	108 520
×16..........	163 200	146 880	143 600	140 350	138 720	137 080	133 800	130 560	124 030
×14..........	166 600	166 600	149 450	146 600	145 180	143 760	140 900	138 080	132 400
×16..........	190 400	190 400	170 800	167 500	165 900	164 300	161 000	157 800	151 300
×16..........	217 600	217 600	217 600	194 700	193 000	191 400	188 200	184 900	178 400

Table IV. Safe Loads in Pounds for Chestnut, Hemlock, Short-Leaf Yellow-Pine and White-Pine Columns, Round and Square

Size of column in inches	Length of column in feet							
	8	10	12	14	15	16	18	20
4×6..........	13 680	12 600	11 520	.....	.....	.....	.....	.....
5½ round.....	14 700	13 930	13 160	12 370	.....	.....	.....	.....
6×6..........	22 630	21 600	20 520	19 440	18 900	18 360	.....	.....
6×8..........	30 240	28 800	27 360	25 920	25 200	24 480	.....	.....
6×10..........	37 800	36 000	34 200	32 400	31 500	30 600	.....	.....
7½ round.....	28 900	27 850	26 780	25 720	25 190	24 660	.....	.....
8×8..........	42 240	40 768	39 360	37 880	37 120	36 480	35 000	.....
8×10..........	52 800	50 960	49 200	47 360	46 400	44 600	43 760	.....
8×12..........	63 360	61 152	59 040	56 830	55 680	54 720	52 500	.....
9½ round.....	47 960	46 440	45 160	43 740	43 100	42 400	41 120	.....
10×10..........	75 000	66 000	64 200	62 400	61 500	60 600	58 800	57 000
10×12	90 000	79 200	77 040	74 880	73 800	72 720	70 560	68 400
10×14..........	105 000	92 400	89 880	87 360	86 100	84 840	82 320	79 800
11½ round....	77 925	69 820	68 160	66 490	65 770	64 833	63 170	61 600
12×12..........	108 000	108 000	95 040	92 880	91 700	90 700	88 560	86 400
12×14..........	126 000	126 000	110 800	108 300	107 000	105 840	103 300	100 802
12×16..........	144 000	144 000	126 700	123 800	122 300	120 900	118 000	115 200
14×14..........	147 000	147 000	147 000	129 300	128 100	127 000	124 400	121 900
16×16..........	192 000	192 000	192 000	192 000	170 500	168 900	166 100	163 000
18×18..........	243 000	243 000	243 000	243 000	243 000	217 000	213 800	210 600
20×20..........	300 000	300 000	300 000	300 000	300 000	300 000	267 600	264 000

Table V. Safe Loads in Pounds for Cedar, Cypress, Redwood, Norway Pine and Whitewood Columns, Round and Square

Size of column in inches	Length of column in feet							
	8	10	12	14	15	16	18	20
4×6..........	11 520	10 550	9 800	8 700	.....	.....	.....	.....
5½ round.....	12 350	11 730	11 180	10 490	.....	.....	.....	.....
6×6..........	19 080	18 216	17 352	16 490	16 050	15 620	.....	.....
6×8..........	25 440	24 290	23 140	21 980	21 400	20 830	.....	.....
6×10..........	31 800	30 360	28 920	27 480	26 760	26 040	.....	.....
7½ round.....	24 220	23 380	22 540	21 660	21 260	20 820	.....	.....
8×8..........	35 450	34 300	33 150	32 000	31 420	30 850	29 700	.....
8×10..........	44 320	42 480	41 440	40 000	39 230	38 560	37 120	.....
8×12..........	53 180	51 450	49 730	48 000	47 140	46 270	44 544	.....
9½ round.....	40 000	39 000	37 860	36 800	36 230	35 730	34 670	.....
10×10..........	62 500	55 400	53 960	52 520	51 800	51 080	49 640	48 200
10×12..........	75 000	66 480	64 800	63 000	62 160	61 300	59 570	57 840
10×14..........	87 500	77 560	75 600	73 500	72 520	71 510	69 500	67 480
11½ round....	64 930	58 390	57 140	55 800	55 170	54 550	53 100	51 950
12×12..........	90 000	90 000	79 780	78 000	77 180	76 320	74 590	72 860
12×14..........	105 000	105 000	93 170	91 050	90 050	89 000	87 020	85 000
12×16..........	120 000	120 000	106 300	104 000	102 900	101 700	99 400	97 150
14×14..........	122 500	122 500	110 350	108 350	107 400	106 400	104 460	102 300
16×16..........	160 000	160 000	160 000	143 870	142 590	141 570	139 260	136 960
18×18..........	202 500	202 500	202 500	202 500	183 060	181 760	179 170	176 580
20×20..........	250 000	250 000	250 000	250 000	250 000	250 000	224 500	221 200

8. Eccentric Loading of Wooden Columns

eneral Principles. When the load on a short column or post is not axial, t is, when the column supports a girder on one side only, or when the weight n one girder is much more than that from the others, the load is said to be ntric, and the distance from the point of application of the load to the axis he column, denoted by p, is called the ECCENTRICITY of the load. It is evi- t that the stress in the column will increase with p, and that the total unit s S, on the side of the column in which the pression is the greatest, will be greater than for equal axial load.

mula for Eccentric Load. Suppose the ntric load to be applied as shown in Fig. 1, n the sectional area of the required square or ngular column may be computed by the fol- ing formula (See, also, page 486):

he sectional area of the column in square hes is

$$A = (P + P_1)/S + 6\,P_1 p/Sd \qquad (6)$$

hich A = sectional area in square inches
P = concentric load on column in pounds
P_1 = eccentric load in pounds
S = safe stress in pounds per square inch
p = distance from axis of column to center of bearing in inches
d = side of column parallel with girder, in inches

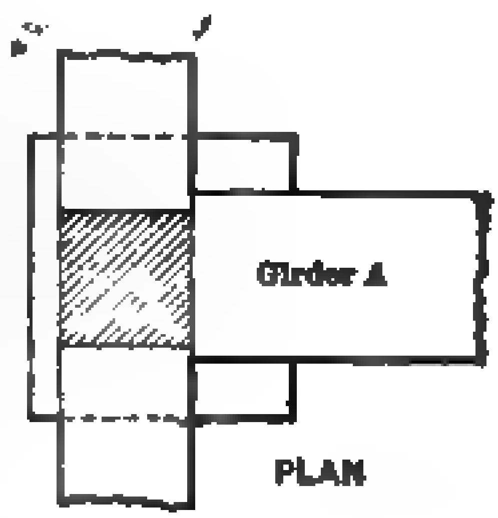


Fig. 1. Eccentric Load on Wooden Column

ssuming the value of S, the probable ratio of ide to the length of the column should be taken account. Thus if it is probable that the length not exceed 12 times the side, both being sured in inches, for oak, long-leaf yellow-pine or glas-fir columns, or 10 times the side for other ds, then the value of S for short columns may taken. If the ratio will probably be greater this, then the probable ratio should be roughly mated and S computed for that ratio by the ula given for columns more than 10 or 12 diameters in length, as noted in eding paragraphs.

mple 3. The lower post in Fig 1 supports a total load on its cap-plate 000 lb, including the reaction of 12 000 lb from girder A What should be ize of the column if made of Douglas fir and if 12 ft in height?

ution. As it is probable that the column will have to be 10 in square S may aken from Table I. With a factor of safety of 5, this is equal to 4 500/5 = lb per sq in. P_1 = 12 000 lb, d = 10 in and p, the distance from the axis of column to the center of bearing of the girder = 7 in. Then from Formula (6), sectional area of the column is

$$A = \frac{60\,000}{900} + \frac{6 \times 12\,000 \times 7}{900 \times 10} = 66.6 + 56 = 122.6 \text{ sq in,}$$

equivalent to a 12 by 12-in square column. From Table III, it may be that an 8 by 10-in column concentrically loaded will carry almost 60 000 lb.

Hence, the eccentric load from the girder increases the dimensions of the cro
section of the column from 8 by 10 to 12 by 12 in.

For wooden columns having a length of over 12 diameters for Douglas fir a
spruce and over 10 diameters for other woods the safe load per square i
should be found by using Formulas (3), (4) or (5).

Example 4. What size will be required for a white-oak column, 14 ft in len
to carry a total load of 56 000 lb, 16 000 lb of which act as an eccentric load fr
a girder, the distance from the center of bearing of the girder to the column be
2 in.

Solution. From Table II, it is probable that at least a 10 by 10-in colu
will be required, so that S must be calculated by Formula (2).

$$S = 1\,000 - 10 \times \frac{\text{length in in}}{\text{breadth in in}}$$

Substituting,
$$S = 1\,000 - 10 \times \frac{168}{10} = 832 \text{ lb per sq in}$$

Substituting in Formula (6),

$$A = \frac{56\,000}{832} + \frac{6 \times 16\,000 \times 7}{832 \times 10} = 68 + 80 = 148 \text{ sq in}$$

equivalent to a 12 by 12-in column.

6. Metal Caps and Bolsters for Wooden Columns

Use of Metal Post-Caps. Whenever wooden posts are used in tiers,
above another, each post except the top one should have an iron cap-p
and the upper post should be set on the cap of the post below and not o
girder. Where a wooden post supports a girder, only, a wooden bolster ma
used in place of the cap but modern approved metal post-caps are always
erable to wooden bolsters. Details of post-caps and bolsters are show
Chapter XXII.

7. Crushing of Wood Perpendicular to the Grain

Safe Unit Stresses. The bearing of wooden girders, the ends of col
resting on girders, and washers on truss-rods, should be proportioned so
the quotient obtained by dividing the load by the bearing area will not e
the safe unit stresses given in Table VI.

Table VI. Safe Loads for Wood Perpendicular to the Grain

Kind of wood	Safe loads, lb per sq in	Kind of wood	Saf loa lb per
White oak	500	Cedar	2
Long-leaf yellow pine	350	Spruce	2
Douglas fir	200	Hemlock	1
Norway pine	200	Cypress	2
White pine	200	Redwood	1
Short-leaf yellow pine	250	Chestnut	2

8. Cast-Iron Columns*

Cast-Iron Versus Steel Columns. Although steel is being used more and more every year for columns in buildings, it will probably never entirely supplant cast iron for buildings of moderate height. For skeleton construction, however, when the height of the building exceeds twice its width, riveted steel columns, with riveted connections with the beams and girders, are unquestionably better; but for the larger proportion of buildings of moderate height, cast iron will probably have the preference for some time to come because it is more economical.

Advantages of Cast-Iron Columns. The commercial advantages which favor the use of cast-iron columns are these:

(1) **Cheapness.** As far as the cost of production is concerned, cast iron is cheaper than steel. This consideration alone often decides in favor of its employment. The raw material is easily transported as pig iron is sometimes brought over as ballast; so that competition with foreign countries keeps down the price.

(2) **Availability.** Cast iron is the most available form of iron. An iron-foundry requires no very elaborate plant, scarcely more than a few furnaces and sand molds, and moreover, no very extensive capital is required to operate it; consequently, the product may be obtained in almost any locality. In rolling-mills, on the contrary, the machinery must be very heavy in order that it may overcome the enormous pressure due to the resistance of the steel in rolling, and to operate it requires a great amount of power.

(3) **Readiness with Which it May be Obtained.** Columns and other structural members if made of cast iron may be obtained much more quickly than if made of steel. After the pattern has once been prepared, a dozen castings may be made almost as quickly as one, and with but very little extra cost, except that of the additional raw material and the expense of remelting it. On the other hand, columns and girders built up of rolled sections take considerably longer to make. Sections can be punched only one at a time, and if they do not happen to be of some standard length, they must be cut and fitted separately before all can be finally riveted together.

(4) **Physical Advantages.** Cast iron is one of the best materials to resist compression, its ultimate compressive strength being as high as 80 000 lb per sq in and even higher. Moreover, it can be molded into almost any desired form, and lugs, brackets and flanges may be cast upon a column all in one piece thus greatly simplifying the cost of erection. In fact, the ease with which the beam and girder-connections can be made is one of the chief reasons for the popularity of cast iron. Finally, it resists fire better than steel and it corrodes less easily. Because of this, its use is advocated by many for the wall columns of skeleton structures, as these columns are particularly liable to corrode. In the Mutual and Manhattan Life Insurance Company's Buildings in New York City, for example, the wall columns are of cast iron, whereas the interior ones are of steel.

Disadvantages of Cast-Iron Columns. The disadvantages of cast iron for columns are as follows:

(1) **Physical Disadvantages.** Cast iron is hard and brittle and cannot be punched or riveted, as the blows required in driving the rivets would very likely fracture the castings; consequently, all connections have to be made with bolts. A bolted connection even under the most favorable conditions is not very rigid,

* See, also, Chapter XIII, pages 445 to 447.

as it allows more or less lateral movement, which, in the case of a tall, nar building, is a serious matter. Owing to the low tensile and shearing strength cast iron, the brackets supporting beams and girders are unreliable and requ great skill in designing. (See pages 445 to 447.)

(2) **Defects in the Castings and the Difficulties of Thorough Inspection.** T castings themselves are subject to a number of serious defects. In the first pl owing to the shifting or floating of the cores, variations in the thickness of hol castings are not infrequent; in fact, it is very difficult to avoid them even w the best care and workmanship. Moreover, there are apt to be concea cavities, blow-holes or honeycomb, and foreign substances, such as cinders sand, any of which may be on the inside of a casting, where a careful exami tion often fails to reveal them. The most critical condition, however, is t due to the uneven contraction of the metal during the process of cooling, thin parts of the casting cooling and contracting more quickly than the th ones, thereby giving rise to INITIAL STRESSES, at times of sufficient inten to fracture the casting before any external loads whatever have been plac upon it. In many cases this trouble is due to faulty designing or to carelessn in handling the molds; yet, even under the most favorable conditions, it is difficult to secure equal radiation from the molds in all directions that castin entirely free from inherent shrinkage-stresses are probably seldom produced

9. Design of Cast-Iron Columns

Common Forms of Cast-Iron Columns. Figs. 2, 3 and 4 show so common forms of cross-sections of cast-iron columns. Columns of circular rectangular cross-sections are always made hollow and the diameter should made as large as possible, within reason of course; because of two colum

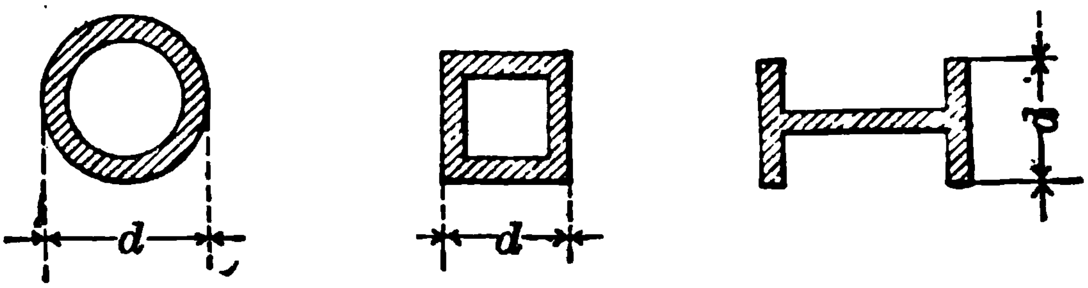


Fig. 2 Fig. 3 Fig. 4

Figs. 2, 3 and 4. Cross-sections of Cast-iron Columns

having the same area of cross-section, the one which, within certain limits, the greater diameter, and consequently the thinner shell, is the stronger. T maximum thickness of shell is 1¾ or 2 in, because of the difficulty of keep the core from shifting in columns of greater thickness; and the minimum thi ness is ¾ in. The latter is a requirement of most municipal building co As the maximum limit of diameter, 16 in may be taken; beyond this, built steel columns can be used to better advantage and are less expensive. T minimum diameter permitted by most building codes is 5 in, and the unsuppor length of the column is limited to 20 times the least diameter.

Hollow, Cylindrical Cast-Iron Columns. The most economical form cross-section, as far as structural requirements are concerned, is the HOLL CIRCLE (Fig. 2). This form is generally used for interior columns; but for terior columns it is not so desirable, because such columns cannot be bonded walls so readily, and do not present the same facilities for the design of the b and girder-connections as columns having the other forms of cross-secti

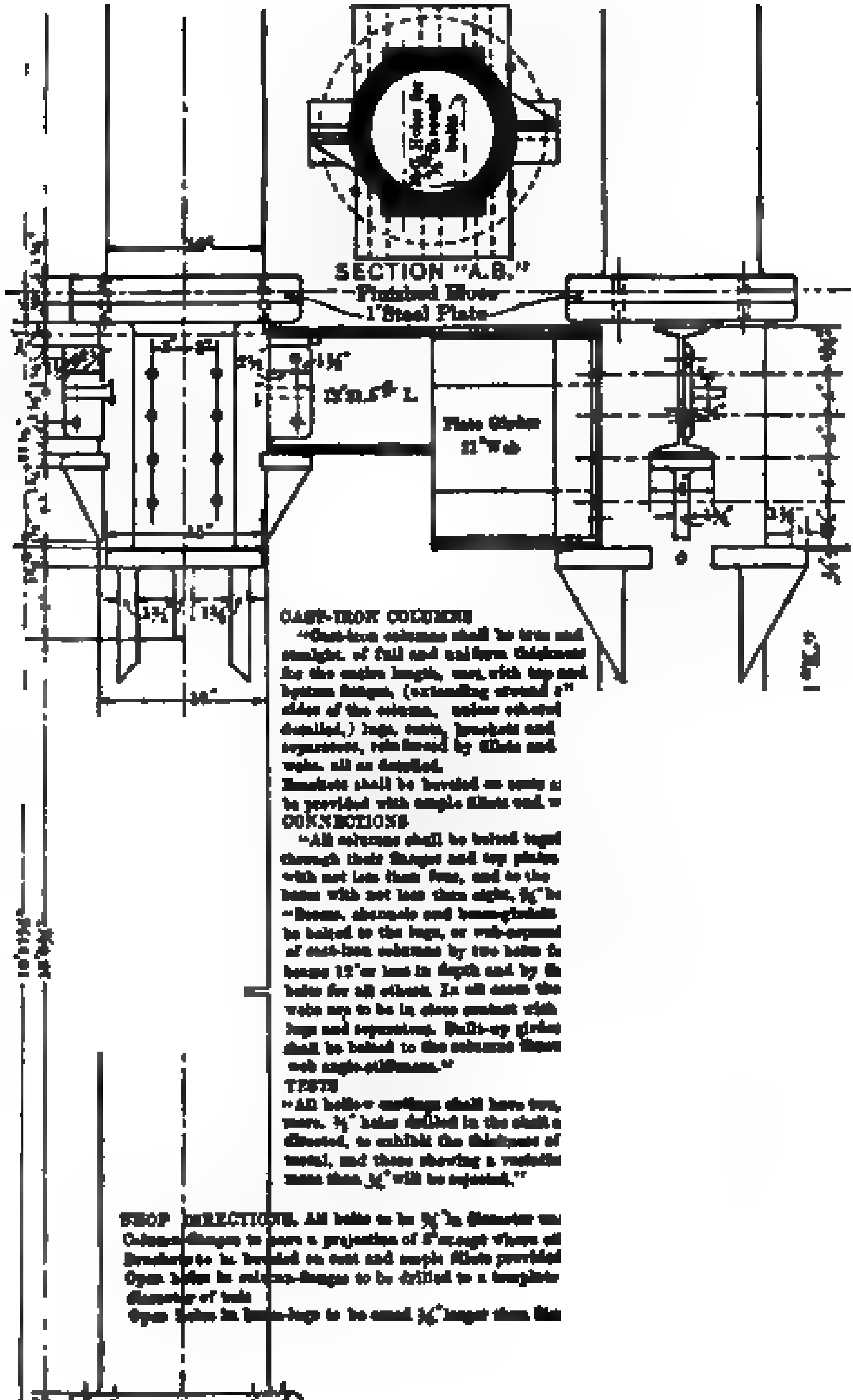


Fig. 5. Connections for Cylindrical Cast-iron Columns

Typical Connections for a Cylindrical Cast-Iron Column. Fig. 5 ... details of a cylindrical cast-iron column with typical beam and girde... ...ctions, dimensions and specification-notes. (See, also, details of connec... ...ackets, base-plates, etc., for cylindrical columns in Chapter XIII, Figs ... 7 and Table III of same chapter.)

Cast-Iron Columns with Hollow-Square Cross-Section. The colum next in point of economy of cross-section are tho with the HOLLOW-SQUARE cross-section (Fig ? They are generally used for wall columns becau it is easier to bond them into the masonry than they had a circular section. Columns of holl rectangular cross-section of unequal sides are som times found to be more available than those of squa section.

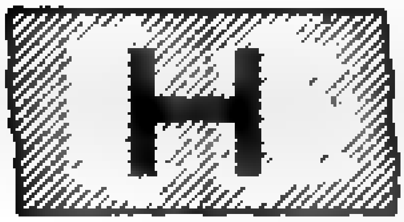


Fig. 6. H-shaped Cross-section of Cast-iron Column

The H-Shape Column (Fig. 4) ranks third regard to economy of material. It is particularly well adapted for wall colum in skeleton construction for the following reasons:

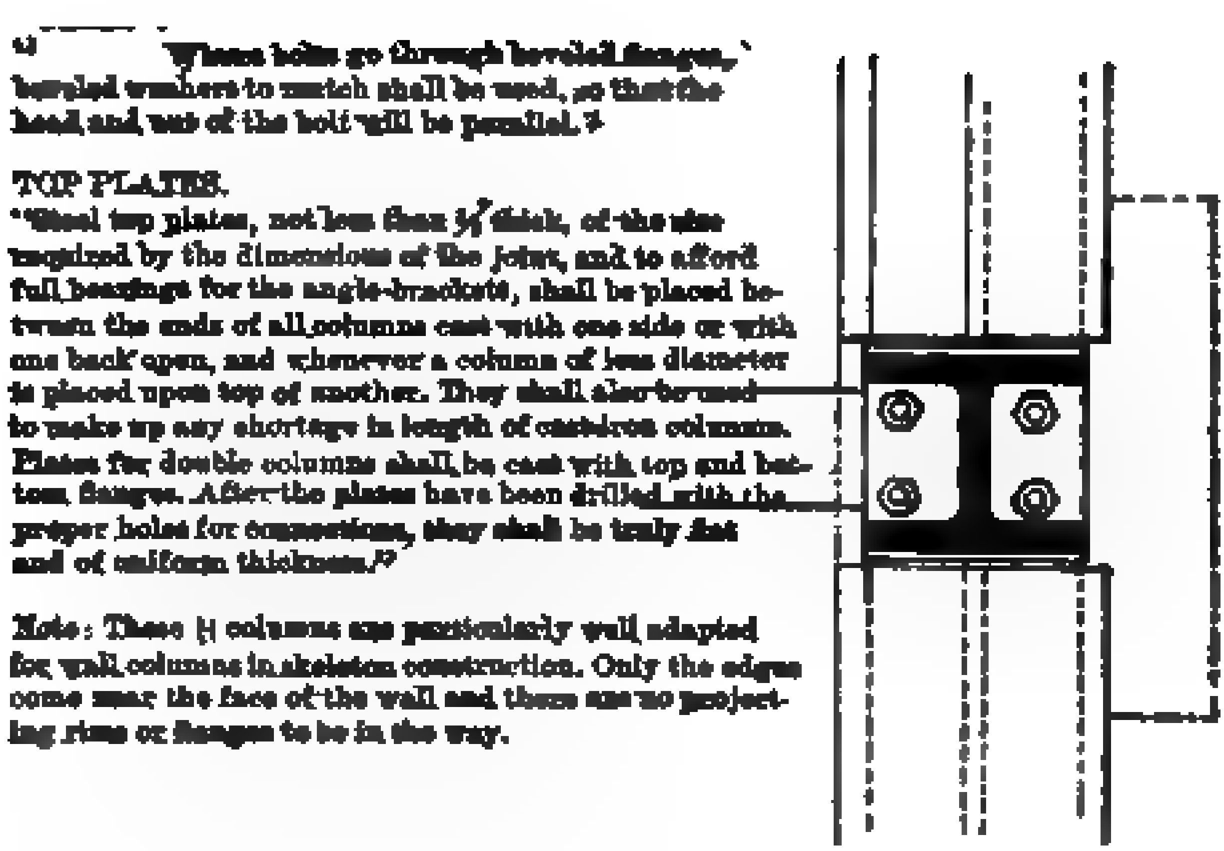


Fig. 7. Connections for H-shaped Cast-iron Column

(1) Being entirely open, with both the interior and exterior surfaces expo any inequalities in thickness can be readily discovered and the thickness is

ily measured, thus obviating any necessity for drilling, and rendering the spection of the columns much easier.

(2) The entire surface of the column may be protected by paint.

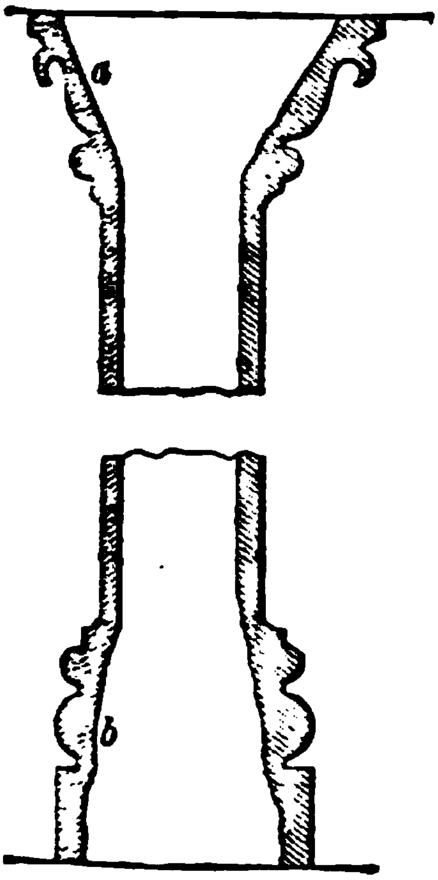
(3) When built in brick walls the masonry fills all voids, so that no open space left; and if the column is placed as shown in Fig. 6, only its edges come near e face of the wall.

(4) Lugs and brackets can be cast on such columns more readily and effectively an on cylindrical columns, especially for wide and heavy girders, and the nections do not require projecting flanges, which are often in the way on lindrical columns.

(5) An eccentric load may be applied to the web where its effect is less and here it is more evenly distributed than when it is applied to the outer rim or ell.

Details of connections and brackets for H-shaped cast-iron columns are shown Fig. 7.

Details of Connections of Cast-Iron Columns. The bearings of a cast-iron column should always be faced true to the axis of the column, and the columns should be bolted together by four ¾-in bolts for columns 10 in in diameter or less, and by six bolts for 12-in and larger columns. Faced plates, as shown in Fig. 5, are inserted between the flanges of columns to make up for any shortage in length and also when a column of smaller diameter is placed over one of greater diameter. For convenience in erecting columns, the joint is generally placed just above the beams or girders supported by the columns.



g. 8. Cast-iron Column with ap and Base. Wrong Method

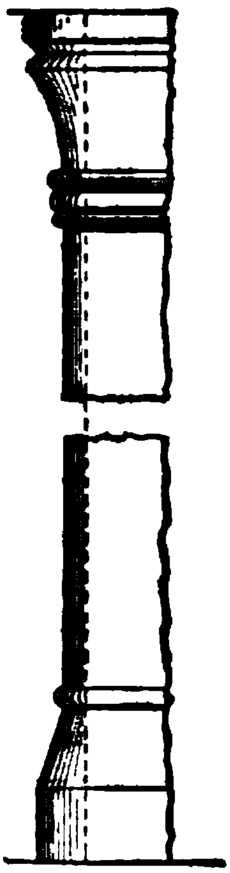


Fig. 9. Cast-iron Column with Cap and Base. Slight Projections

Projecting Caps and Bases. A column with ornamental cap and base should ver be cast as shown in Fig. 8, that is, if it is to support load. In every bearing column, the core should extend a straight line from end to end. Plain molded caps and bases may be cast id as in Fig. 9; but if more ornamental caps are desired, or heavy projecting es, they should be cast separately and attached to the straight columns by ews.

10. Strength of Cast-Iron Columns. Formulas

Formulas for Cast-Iron Columns. The ULTIMATE RESISTANCE of cast iron crushing is generally taken at 80 000 lb per sq in, and for posts, pintels, etc., re the length is not more than six times the diameter or breadth, it will lly be safe to assume a WORKING STRENGTH of six tons per square inch of al. For longer posts or columns, the strength is affected by the ratio of

length to diameter, but to just what extent is not known with absolute certai hence all formulas for columns must be more or less theoretical. The co quence is that while a great many formulas have been published, there is that is universally accepted. The two following Formulas * (7) and (8), for many years more commonly adopted than any others, as they appear agree as well as any with actual tests.

Formula for Hollow, Cylindrical, Cast-Iron Columns with Square End Ultimate strength, in pounds.

$$= \text{metal-area} \times \left[80\,000 + \left(1 + \frac{\text{sq of length in in}}{800 \times \text{sq of diam in in}} \right) \right]$$

or

$$\text{Ultimate strength, in pounds} = \frac{80\,000\,A}{1 + l^2/800\,d^2}$$

in which A is the area of the cross-section in square inches.

* The tables in the handbook of the Cambria Steel Company (1913) are ba Formulas (7) and (8), and they were adopted in some building laws. They are base the form of Gordon's formula, which, in turn, is Rankine's formula with d, the di or least lateral dimension, substituted for r, the least radius of gyration of the cro tion. Rankine's formula is sometimes referred to as Gordon's formula. The obtained by these formulas will be slightly in excess of those given in the old C building law (see tabulation in this foot-note), and considerably less than those per by the former building law of New York City, $S = 11\,300 - 30\,l/r$. (Present $S = 9\,000 - 40\,l/r$.

In 1898 Professor W. H. Burr made an analysis of the results of a number of ments on full-size, hollow, cylindrical cast-iron columns made at the Watertown A Mass., and at Phœnixville, Pa., and by plotting the results found that a straig formula having the equation $S = 30\,500 - 160\,l/d$, in which S is the ultimate st of the metal per square inch of column-area, represented the average of the results. With a factor of safety of 4 this would become $S = 7\,625 - 40\,l/d$ and factor of safety of 5, $S = 6\,100 - 32\,l/d$.

According to Professor Burr's analysis the values for S given in the fourth col Table VII represent a factor of safety of a little over 4 for $l/d = 20$, and of near $l/d = 36$.

Formulas for finding the value of S according to the former codes of Chicago and

Cylindrical columns		Rectangular columns	
Old Chicago Code	Old Boston Code	Old Chicago Code	Old Boston
$\dfrac{10\,000}{1 + \dfrac{l^2}{600\,d^2}}$	$\dfrac{10\,000}{1 + \dfrac{l^2}{800\,d^2}}$	$\dfrac{10\,000}{1 + \dfrac{l^2}{800\,d^2}}$	$\dfrac{10\,000}{1 + \dfrac{l^2}{1066}}$

The former New York City Building Code Formula was

$$S = 11\,300 - 30\,l/r$$

Compared with the results of tests that have been made on full-size cast-iron col has been shown that while in Chicago a factor of safety of 8 was allowed, the actu of safety was a little over 4, that in Boston it was slightly under 4, while in New was a trifle over 6. The formula in the new (1916) Chicago code is $S = 10\,000 -$ while the new (1915) Boston code gives the values of S for l/r from 10 to 70.

A series of tests on full-size cast-iron columns and brackets was made under th tion of Stevenson Constable, in December, 1897, a report of which, with illust may be found in the Engineering Record for January 8 and 22, 1898.

Formula for Hollow, Rectangular, Cast-Iron Columns with Square Ends

imate strength, in pounds

$$= \text{metal-area} \times \left[80\,000 \div \left(1 + \frac{\text{sq of length in in}}{1\,067 \times \text{sq of least side in in}}\right)\right] \quad (8)$$

$$\text{Ultimate strength, in pounds} = \frac{80\,000\,A}{1 + l^2/1\,067\,d^2}$$

hich A is the area of the cross-section in square inches

Formula for Solid, Cylindrical, Cast-Iron Columns

imate strength, in pounds

$$= \text{metal-area} \times \left[80\,000 \div 1 + \frac{\text{sq of length in in}}{266 \times \text{sq of diam in in}}\right] \quad (9)$$

$$\text{Ultimate strength, in pounds} = \frac{80\,000\,A}{1 + l^2/266\,d^2}$$

hich A is the area of the cross-section in square inches.

For H-shaped columns use formula (7), taking d as the least side.

The SAFE LOAD is generally taken at one-eighth of the ultimate strength or aking-load.

ccentric Loading. Cast-iron columns should not be loaded with a heavy, NTRIC LOAD, that is, a load applied on one side of the column without a responding load on the other side, as cast iron is unable to resist very great ding stresses. (See, also, eccentric loading of wooden and steel columns, es 453 and 485.)

11. Tables of Safe Loads for Cast-Iron Columns. Examples

xplanation of Tables. As the allowable pressure PER SQUARE INCH OF AL depends upon the ratio of length to diameter, without regard to actual ensions (that is, it would be the same for a column 6 in in diameter and 12 ft g as for one 8 in in diameter and 16 ft long), it is practicable to prepare a e which will give the value of the terms of Formulas (7) and (8) inclosed in ckets for all ratios of diameter to length, and thus simplify very much the putation for any particular column. Table VII has been computed by ns of Formulas (7) and (8) for ratios of length to diameter varying from 8 6, and the same result will be obtained by using the values given in this e as by using the corresponding formula. To use this table it is only neces- to divide the length of the column by the least thickness or diameter, both ches, and opposite the number in the first column of the table coming nearest e quotient, find the SAFE STRENGTH PER SQUARE INCH for the column. load is multiplied by the METAL-AREA in the cross-section of the column the result is the SAFE LOAD for the column. Examples (5) and (6) will illus- the use of Tables VII to X.

ample 5. What is the safe load for a 10-in hollow, cylindrical cast-iron n, 15 ft long, the shell being 1 in thick?

ution. In this case the ratio l/d, which is the length of the column divided e diameter, both in inches, is 18, and opposite 18 in Table VII the safe per square inch for a cylindrical column is found to be 7 117 lb. The metal- f the column, from the table of areas on pages 42 and 463, is equal to the f a 10-in circle minus the area of an 8-in circle, or, 78.53 − 50.26 = 28.27 Multiplying these two together, for the safe load of the column the result 7 sq in × 7 117 lb per sq in = 201 917 lb, or about 100.5 tons.

Tables VIII, IX and X. To still further facilitate computations, Tabl VIII, IX and X, have been prepared, which give at a glance the safe loa based on a factor of safety of 8, for columns of the more common sizes a lengths. For lengths between those given in the tables sufficiently accura results may be obtained by interpolation. For any other factor of safet multiply the safe load given in the table by 8, and divide by the new fac of safety.

Example 6. What is the safe load for a 9-in hollow, cast-iron column square cross-section 12 ft long, the shell being 1 in thick?

Solution. From Table IX, the safe load is 129 tons. The same result m be obtained by using Table VII. The ratio l/d in this case is $144/9 = 16$ a the corresponding safe load in pounds per square inch is 8 064. The area the column is 32 sq in. Hence, the safe load is 32 sq in × 8 064 lb per sq in 258 048 lb, or 129 tons, which agrees with the safe load given in Table IX the same column.

Table VII. Breaking-Loads and Safe Loads in Pounds per Square Inch for Hollow, Cylindrical and Hollow, Rectangular, Cast-Iron Columns

Calculated by Formulas (7) and (8)

Length in inches divided by external breadth or diameter	Breaking-weight in pounds per square inch		Safe loads in pounds per square inch. Safety-factor 8	
	Cylindrical	Rectangular	Cylindrical	Rectangula
8	74 074	75 470	9 259	9 433
9	72 661	74 350	9 082	9 293
10	71 110	73 126	8 888	9 140
11	69 505	71 870	8 688	8 983
12	67 800	70 487	8 475	8 811
13	66 060	69 084	8 257	8 635
14	64 257	67 567	8 032	8 446
15	62 450	66 060	7 806	8 257
16	60 606	64 516	7 576	8 064
17	58 780	62 942	7 347	7 867
18	56 940	61 360	7 117	7 670
19	55 134	59 745	6 892	7 468
20	53 333	58 180	6 666	7 272
21	51 580	56 610	6 447	7 076
22	49 843	55 020	6 230	6 877
23	48 163	53 470	6 020	6 684
24	46 512	51 950	5 814	6 494
25	44 918	50 440	5 614	6 305
26	43 360	48 960	5 420	6 120
27	41 862	47 530	5 233	5 940
28	40 404	46 110	5 050	5 764
29	39 000	44 742	4 875	5 592
30	37 647	43 390	4 706	5 424
31	36 347	42 080	4 543	5 260
32	35 090	40 816	4 386	5 102
33	33 884	39 580	4 235	4 947
34	32 720	38 380	4 090	4 797
35	31 608	37 244	3 951	4 655
36	30 534	36 120	3 817	4 515

Table VIII. Safe Loads in Tons of 2 000 pounds for Hollow, Cylindrical, Cast-Iron Columns with Square Ends

Based on Formula (7). Safety-factor 8

[illegible]	Thickness, in	Length of column in feet											
		6	8						20				
5	¾	39	34							.	..	10 0	31 3
	⅞	45	38						..	....	...	11 3	35 3
5½	¾	46	40						...	....	...	11.2	35 0
	⅞	52	46	40	34	29	.	.	.	..	.	12 7	39 7
6	¾	52	47	41	36	31	27	24	...	...	.	12 4	38 7
	⅞	60	53	47	41	36	31	27	..	....	...	14 1	44 0
	1	66	[illegible]	[illegible]	45	39	34	30	...	..		[illegible]	[illegible]
7	¾	65	[illegible]	[illegible]	48	43	38	34	..	...			
	⅞	74	[illegible]	[illegible]	55	49	43	38	..	...	.		
	1	83	[illegible]	[illegible]	61	54	48	43	..	....	.		
8	¾	78	[illegible]	[illegible]	61	55	50	45	40	36	[illegible]		
	⅞	89	[illegible]	[illegible]	70	63	57	51	46	41	[illegible]		
	1	100	[illegible]	[illegible]	79	71	64	58	52	47	[illegible]		
9	⅞	103	[illegible]	[illegible]	85	80	71	65	59	54	[illegible]		
	1	117	[illegible]	[illegible]	95	90	80	73	67	61	[illegible]		
	1⅛	129	[illegible]	[illegible]	105	99	89	81	74	67	[illegible]		
10	⅞	118	[illegible]	[illegible]	100	93	86	79	73	67	[illegible]		
	1	133	[illegible]	[illegible]	112	105	97	89	82	76	[illegible]		
	1⅛	[illegible]	[illegible]	[illegible]	125	116	107	99	91	84	[illegible]		
	1¼	[illegible]	[illegible]	[illegible]	136	127	118	109	100	92	[illegible]		
11	1	[illegible]	[illegible]	[illegible]	129	122	114	106	98	91	[illegible]		
	1⅛	[illegible]	[illegible]	[illegible]	144	135	126	118	109	101	[illegible]		
	1¼	[illegible]	[illegible]	[illegible]	158	148	139	129	120	111	[illegible]		
	1⅜	[illegible]	[illegible]	[illegible]	171	161	151	140	130	121	[illegible]		
12	1⅛	[illegible]	[illegible]	[illegible]	163	154	146	137	128	120	[illegible]		
	1¼	[illegible]	[illegible]	[illegible]	179	170	160	150	141	132	[illegible]		
	1⅜	[illegible]	[illegible]	[illegible]	194	184	174	163	153	143	[illegible]		
	1½	[illegible]	[illegible]	[illegible]	210	199	187	176	165	154	[illegible]		
13	1⅛	[illegible]	[illegible]	[illegible]	182	174	165	156	147	138	[illegible]		
	1¼	[illegible]	[illegible]	[illegible]	200	191	181	172	162	152	[illegible]		
	1⅜	[illegible]	[illegible]	[illegible]	218	208	197	187	176	166	[illegible]		
	1½	[illegible]	[illegible]	[illegible]	235	224	213	201	190	179	[illegible]		
14	1¼	[illegible]	[illegible]	[illegible]	221	212	203	193	183	173	[illegible]		
	1⅜	[illegible]	[illegible]	[illegible]	241	231	221	210	199	189	[illegible]		
	1½	[illegible]	[illegible]	[illegible]	260	250	238	227	215	204	[illegible]		
	1⅝	[illegible]	[illegible]	[illegible]	279	268	256	243	231	219	[illegible]		
15	1⅜	[illegible]	[illegible]	[illegible]	264	254	244	234	223	212	[illegible]		
	1½	[illegible]	[illegible]	[illegible]	285	275	264	252	241	229	[illegible]		
	1⅝	[illegible]	[illegible]	[illegible]	306	295	283	271	259	246	[illegible]		
	1¾	[illegible]	[illegible]	[illegible]	327	315	302	288	276	263	251		
16	1¼	[illegible]	[illegible]	[illegible]	310	300	290	278	267	255	243		
	1⅜	[illegible]	[illegible]	[illegible]	333	322	311	299	286	273	261		
	1½	[illegible]	[illegible]	[illegible]	356	344	332	319	306	292	279		
	1⅞	[illegible]	[illegible]	[illegible]	423	410	395	380	364	347	332		

Table IX. Safe Loads in Tons of 2 000 Pounds for Hollow, Square and Rectangular, Cast-Iron Columns, with Square Ends

Based on Formula (8). Safety-factor 8

Size, in	Thickness, in	Length of column in feet								Area of metal, sq in	Weight lin ft
		8	10	12	14	16	18	20	24		
4×6	¾	41	34	28	....	....	....	....	....	12.75	39.8
4×8	¾	51	42	35	....	....	....	....	....	15.75	49.2
4×9	¾	56	46	39	....	....	....	....	....	17.25	53.9
4×10	¾	60	50	42	....	....	....	....	....	18.75	58.6
4×12	¾	70	59	49	....	....	....	....	....	21.75	68.0
5×8	¾	64	55	48	41	....	....	....	....	17.25	53.9
	1	81	71	61	53	....	....	....	....	22.00	68.8
5×9	¾	69	60	52	45	....	....	....	....	18.75	58.6
	1	89	78	67	58	....	....	....	....	24.00	75.0
5×10	¾	75	65	57	49	....	....	....	....	20.25	63.3
	1	96	84	73	63	....	....	....	....	26.00	81.3
5×12	¾	86	74	65	56	....	....	....	....	23.25	72.7
	1	111	97	84	72	....	....	....	....	30.00	93.8
6×6	¾	63	57	51	45	40	35	....	....	15.75	49.2
	1	80	72	65	57	51	45	....	....	20.00	62.5
6×8	¾	75	68	60	54	47	42	....	....	18.75	58.
	1	96	87	78	69	61	54	....	....	24.00	75.
6×9	¾	81	73	65	58	51	45	....	....	20.25	63.
	1	104	94	84	75	66	58	....	....	26.00	81.
6×10	¾	87	79	70	62	55	49	....	....	21.75	68.
	1	112	101	91	80	71	63	....	....	28.00	87.
6×12	¾	99	90	80	71	63	55	....	....	24.75	77.
	1	129	116	104	92	81	72	....	....	32.00	100.
6×15	¾	117	106	95	84	74	66	....	....	29.25	91.
	1	153	138	123	109	97	85	....	....	38.00	118.
7×7	¾	80	73	67	61	55	49	44	....	18.75	58.
	1	102	94	85	78	70	63	57	....	24.00	75.
7×9	¾	92	85	77	70	63	57	51	....	21.75	68.
	1	119	109	100	91	82	74	66	....	28.00	87.
7×12	¾	111	102	93	85	77	69	62	....	26.25	82.
	1	144	133	121	110	99	89	80	....	34.00	106.
8×8	¾	95	90	83	77	70	64	59	49	21.75	68.
	1	124	115	107	99	91	83	76	63	28.00	87
	1¼	148	140	129	119	109	100	91	76	33.75	105
8×10	¾	109	103	95	87	80	73	67	55	24.75	77
	1	141	132	122	113	104	95	86	72	32.00	100
	1¼	170	161	148	137	125	115	105	87	38.75	121
8×12	¾	122	115	106	98	90	82	75	62	27.75	86
	1	158	148	138	127	116	107	97	81	36.00	112
	1¼	192	181	167	154	142	130	118	98	43.75	136

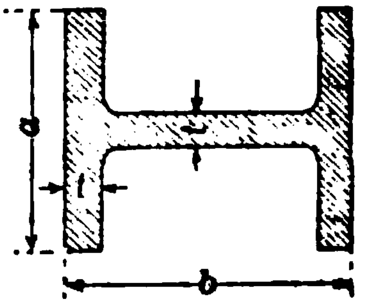
Table IX (Continued). Safe Loads in Tons of 2 000 Pounds for Hollow, Square and Rectangular, Cast-Iron Columns, with Square Ends

Based on Formula (8). Safety-factor 8

Size, in	Thickness, in	Length of column in feet								Area of metal, sq in	Weight, lin ft
		8	10	12	14	16	18	20	24		
8X16	1	193	181	168	155	142	130	119	99	44.00	137.5
	1¼	236	221	206	190	174	159	145	121	53.75	168.0
9X 9	¾	111	106	99	93	86	80	74	63	24.75	77.3
	1	144	137	129	120	112	103	96	85	32.00	100.0
9X12	1	171	162	153	143	133	123	114	97	38.00	118.8
	1¼	209	198	186	174	162	149	138	118	46.25	144.5
9X16	1	207	196	185	173	161	149	138	117	46.00	143.8
	1¼	254	240	226	212	197	182	168	143	56.25	175.8
10X10	1	165	158	150	142	133	125	117	101	36.00	112.5
	1¼	201	193	183	172	162	152	142	123	43.75	136.7
10X12	1	184	176	167	158	148	139	129	112	40.00	125.0
	1¼	224	214	204	192	181	169	158	137	48.75	152.3
10X15	1	211	202	192	181	170	160	149	129	46.00	143.8
	1¼	258	247	235	222	209	195	182	158	56.25	175.8
10X16	1	220	211	200	189	178	167	155	135	48.00	150.0
	1¼	270	258	245	232	218	204	190	165	58.75	183.6
10X18	1	239	228	217	205	193	181	168	146	52.00	162.5
	1¼	293	280	266	251	236	221	207	179	63.75	199.2
10X20	1	257	246	234	221	208	194	181	157	56.00	175.0
	1¼	316	302	287	271	255	239	223	193	68.75	214.9
10X24	1	294	281	267	252	237	222	207	180	64.00	200.0
	1¼	362	346	329	311	292	274	255	221	78.75	246.1
12X12	⅞	183	177	171	164	156	149	141	126	38.90	121.7
	1	207	201	193	185	177	168	159	142	44.00	137.5
	1¼	253	245	236	223	216	206	195	174	53.75	168.0
	1½	296	288	277	265	253	241	228	204	63.00	196.9
12X15	1	235	228	220	211	201	191	181	162	50.00	156.3
	1¼	288	280	269	258	246	234	222	198	61.25	191.4
12X16	1	245	237	228	219	209	199	188	168	52.00	162.5
12X18	1	263	256	246	236	225	214	203	181	56.00	175.0
12X20	1	282	274	264	253	241	229	217	194	60.00	187.5
12X24	1	320	310	299	287	274	260	246	220	68.00	212.5
14X16	1	268	261	254	246	238	229	219	200	56.00	175.0
14X20	1	307	298	290	281	272	261	250	228	64.00	200.0
14X24	1	345	336	326	316	306	294	280	257	72.00	225.0
16X16	1	300	284	278	271	264	256	247	229	60.00	187.5
16X24	1	380	360	352	344	334	324	313	291	76.00	237.5
18X18	1	340	340	320	314	307	299	291	274	68.00	212.5
20X20	1	380	380	361	356	349	342	334	317	76.00	237.5
20X24	1	420	420	399	393	386	378	369	351	84.00	262.5

Table X. Safe Loads in Tons of 2 000 Pounds for H-Shaped, Cast-Iron Columns

Based on Formula (7). Safety-factor 8

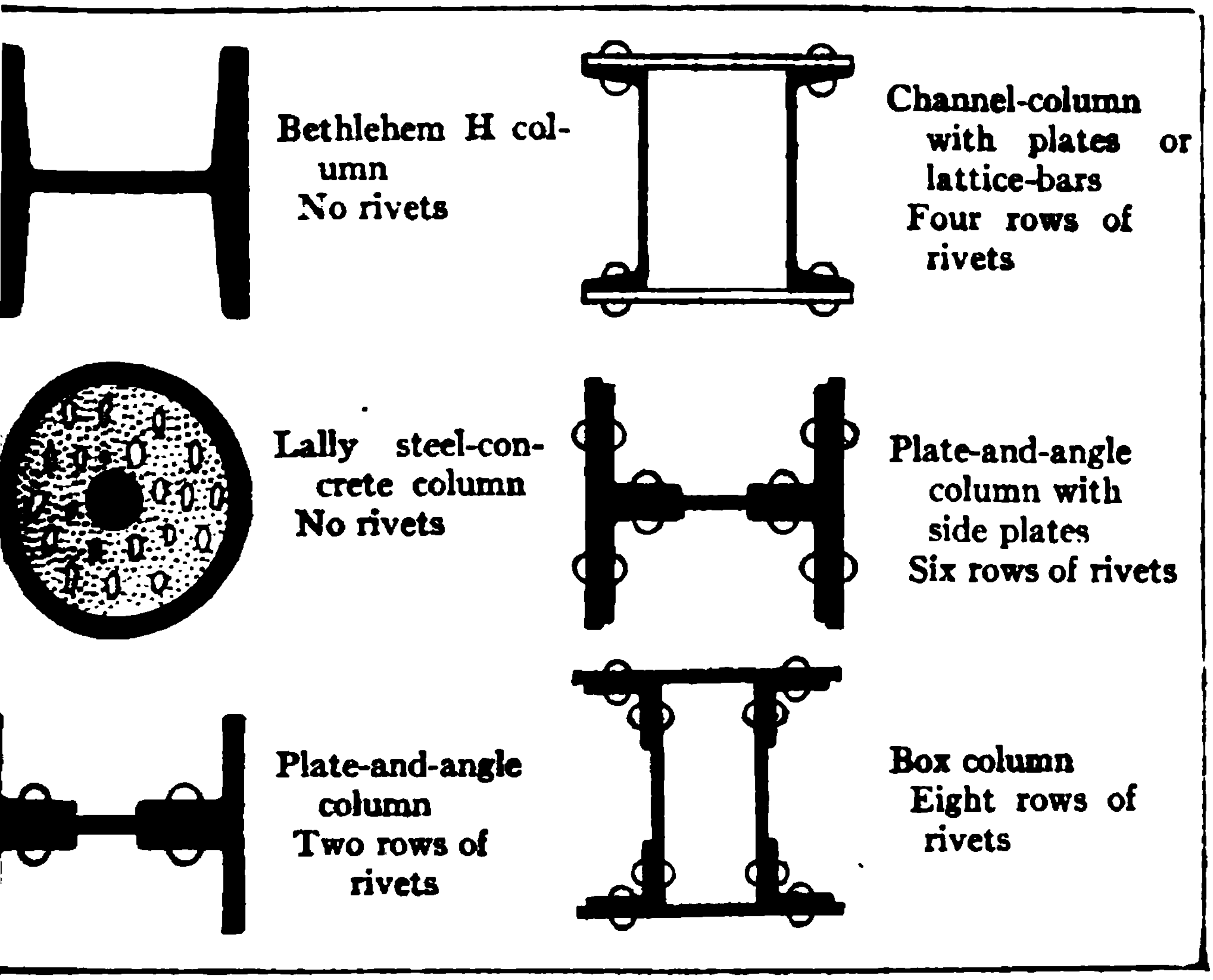


Size, in			Area of metal, in	Length of column in feet							
a	b	t		10	12	13	14	15	16	18	20
6×	6×	¾	12⅜	41	36	33	31				
		1	16	53	46	43	40				
		1¼	19⅜	64	56	52	48				
6×	8×	¾	13⅞	46	40	37	34				
		1	18	60	52	48	45				
		1¼	21⅞	73	63	59	54				
7×	7×	1	19	69	62	58	55	52	49	43	38
		1¼	23⅛	84	75	71	67	63	59	53	46
7×	9×	1	21	76	68	64	61	57	54	48	42
		1¼	25⅝	93	83	79	74	70	66	59	51
8×	8×	¾	16⅞	66	60	57	54	51	49	44	39
		1	22	86	78	74	70	67	64	57	[illegible]
		1¼	26⅞	105	95	91	86	82	78	70	63
8×	10×	1	24	93	85	81	77	73	69	62	56
		1¼	29⅜	114	104	99	94	90	85	76	69
		1½	34½	134	122	117	111	105	100	89	81
9×	9×	1	25	102	94	91	87	83	79	72	66
		1¼	30⅝	125	116	111	106	102	97	89	81
		1½	36	147	136	130	125	120	114	104	95
9×	10×	1	26	106	98	94	90	86	83	75	69
		1¼	31⅞	130	120	115	111	106	101	92	84
		1½	37½	153	142	136	130	125	119	108	99
10×	10×	1	28	118	111	107	103	99	95	88	81
		1¼	34⅜	145	136	131	127	122	127	108	100
		1½	40½	171	160	155	149	144	138	128	117
		1¾	46⅜	196	184	177	171	165	158	146	134
10×	12×	1	30	127	119	115	111	106	102	94	87
		1¼	36⅞	156	146	141	136	131	126	116	[illegible]
		1½	43½	184	172	166	160	154	148	137	[illegible]
		1¾	49⅞	211	198	191	184	177	170	157	144
		2	56	236	222	214	207	199	191	176	162
12×	12×	1	34	151	144	140	136	132	128	121	113
		1¼	41⅞	186	177	172	167	163	158	149	139
		1½	49½	220	209	203	198	193	187	177	165
		1¾	56⅞	252	241	234	227	221	216	202	189
		2	64	284	271	263	256	249	242	227	213
12×	14×	1¼	44⅜	197	188	183	177	173	168	158	148
		1½	52½	233	222	216	210	204	199	186	174
		1¾	60⅜	268	255	248	241	235	228	214	201
		2	68	302	288	280	272	265	257	241	226
		2¼	75⅜	335	319	310	301	292	285	268	251

12. Types, Forms and Connections of Steel Columns

Use of Steel Columns, Struts, Trusses, etc. Owing to the many ad-
ntages of built-up steel columns over cast-iron columns, especially for all
ildings, and to the great reduction that has taken place in the cost of steel
nstruction, built-up columns are now very extensively used in buildings of
en moderate height; and for skeleton construction, or for buildings exceeding
x stories in height, they are certainly much to be preferred to cast-iron columns.
teel trusses, also, are now much more commonly used in buildings than in
rmer years, so that the architect must have at hand data for designing them
nd for computing their strength. In the following pages the author has en-
avored to cover the subject of columns and struts quite completely, to furnish
ch data as will enable the designer to decide upon the shape of column or
rut it is best to use, and also to determine the sizes and sections of such col-
mns with the least labor.

Types and Forms of Steel Columns. The following are cross-sections
f the majority of steel columns in general use, arranged in the order of their sim-
licity of construction, that is, the number of rows of rivets they require:



Considerations Governing the Selection of Steel Columns. There are
nsiderations other than simplicity of construction which sometimes govern
e selection of a column. Some of the most important of these are explained
the following paragraphs:

(1) Cost and Availability of Material. I beams, channels, plates and angles
the most common commercial sections. They are easily rolled and are
nufactured by all of the large mills. They are reasonable in price and may
obtained promptly in large numbers in any locality where a steel building is
ly to be erected. Patented sections, or the product of one mill, do not, as
le, fulfill these conditions.

(2) **Amount of Labor Required and Facility With Which it can be Performed i Shop and Field.** In the shop the complexity of the column-section and th number of pieces of which it is composed greatly affect the cost of labor. there are numerous small pieces such as lattice-bars, splice-plates, etc., eac of which requires cutting and fitting together, with frequent handling, the co is proportionately great. The cost of a column depends, also, largely upon th number of rivets required and whether they can all be driven by machine s as to avoid the slower and more expensive hand-riveting. The same gener remarks apply to labor in the field; the connections should be as simple a possible, the rivets easy of access and as few in number as is consistent wit strength.

(3) **Simplicity of Connections Between Column and Supported Member** This is quite an important consideration in the design of a large building an sometimes governs the choice of the section to be used. Where there are fo beams to a column, on opposite sides, and all of the same height, a satisfacto connection can be made with almost any section; but where the beams a spaced irregularly, both in regard to position in plan and to height, and whe eccentric loads must be provided for, it is very important that the section the column itself affords as great an opportunity as possible for the connectio of the beams. In this respect, possibly, closed sections are inferior to op sections having a central web.

(4) **Adaptability to Connections Which Transfer Compressive Stress Directly to Axis of Column.** In this respect, also, sections of an open constru tion, in which the girders transmit their loads almost directly to the centr axis of the column, thus avoiding the disadvantage of eccentric loading, a superior to those of a closed construction.

(5) **Adaptability to Changes in Thickness of Metal in Members of Column to Suit Different Loads in Different Stories.** It is not desirable to make t columns carrying the upper floors of a building very small, since the beams a girders supporting the upper floors are usually of the same dimensions as tho of the lower floors and consequently require just as heavy and secure conne tions. It is almost impossible to make such connections with small column and consequently, in order to reduce the area of a column in proportion to lighter load to be carried, it is better to reduce the thickness of the materi used and to keep the general dimensions of the section the same.

(6) **Adaptability to Fire-Proof Covering.** Closed sections in general can more compactly fireproofed than open sections.

General Considerations Affecting the Choice of Steel Columns. It almost impossible to say that any one of the foregoing types of steel colum is superior to the others. Each has its own good points, and the column wh section has theoretically the best distribution of material may not always the best one to use, because of the eccentric loads to be carried, or because other conditions. The choice in most cases will depend upon the personal vie of the designer, as well as upon the local conditions as to cost and manufactu promptness of delivery and the details of the problem. Further descripti of the different columns, and also the special advantages claimed for them, given in the following pages.

Steel-Column Connections. When steel columns were first designed it customary to use cap-plates to connect the story-lengths, and the beams girders often rested upon these plates. In modern practice, however, the umn-joint is generally placed just above the beams and girders for convenie in erection and the plates are often omitted The columns are closely fi

together with milled ends, and splice-plates are riveted to the sides or flanges as shown in the illustrations of typical steel-column details, Figs. 17 and 18. As it is impossible in these pages to include the subject of column-connections in anything but a general way, the only attempt that has been made in this direction is to illustrate common forms of connections that have been used with different kinds of columns. These will be found in the description of columns in the following pages.

Number of Rivets Required. No general rule can be given for the number of rivets and size of the brackets required for column-connections, as the loads to be supported vary in different buildings and in different parts of the same building. The number of rivets required in each connection must therefore be determined by the rules given in Chapter XII for designing riveted joints. Connections for single beams, however, will generally require the same number of rivets as are given for beam-connections (Chapter XV, page 617). The allowable stress for rivets in column-connections is generally taken at 10 000 lb per sq in for single shear and 18 000 or 20 000 lb per sq in for bearing. (See Tables II and III, pages 418 and 419, Chapter XII.)

Spacing of Rivets. Steel columns fail either by deflecting bodily out of a straight line or by the buckling of the metal between rivets or other points of support. Both actions may take place at the same time, but if the latter occurs alone, it may be an indication that the rivet-spacing or the thickness of the metal is insufficient. The rule has been deduced from actual experiments upon riveted columns that the distance between centers of rivets should not exceed, in the line of stress, sixteen times the thickness of metal of the parts joined, with a maximum spacing of 6 in, and that the distance between rivets or other points of support, at right-angles to the line of stress, should not exceed thirty-two times the thickness of the metal. The usual practice in designing columns is to space the rivets the minimum distance on centers at both ends, for a length equal to twice the least dimension of the column, with the maximum spacing of 6 in between.

Steel-Pipe Columns.* Steel-pipe columns are used for interior construction to carry beams and girders supporting floors, walls and chimneys in all classes of buildings, such as tenements and apartment-houses, factories, garages, churches, warehouses, etc. A particular demand for steel-pipe columns is at the angles of show-windows in mercantile buildings. In buildings of moderate height the floor-joists are usually supported by the side walls and the columns have to support only a relatively light wall above. For such places wrought-steel pipes may be advantageously used for the columns. They may be used, also, for the columns supporting the roof of one-story buildings. In the Borough of Brooklyn, New York City, pipe-columns were formerly calculated by the formula $S = 14\,000 - 80l/r$, in which S, l and r have values as explained below for New York and Chicago formula. If the columns are filled with concrete, the area of the cross-section of the concrete is multiplied by 500 and the product added to the load supported by the pipe. (See, also, page 477 and the Tables on page 476). This formula gave a factor of safety of four. New York and Chicago codes now use the formula, $S = 16\,000 - 70l/r$ in which S is the permissible unit fiber-stress, l the length in inches and r the radius of gyration of the cross-section of the pipe. This gives a carrying capacity greater than the former formulas gave. In Philadelphia, pipe-columns are allowed to carry

* Much valuable data relating to steel-pipe columns was furnished the editor-in-chief by P. C. Patterson and J. A. McCullough of the National Tube Company, Pittsburgh, Pa.

about 6% more than is allowed in New York. Where pipe-columns are fille with concrete the cast cap and base are secured to the pipe in each case by con crete which is reinforced internally by a pipe of smaller diameter. Whe

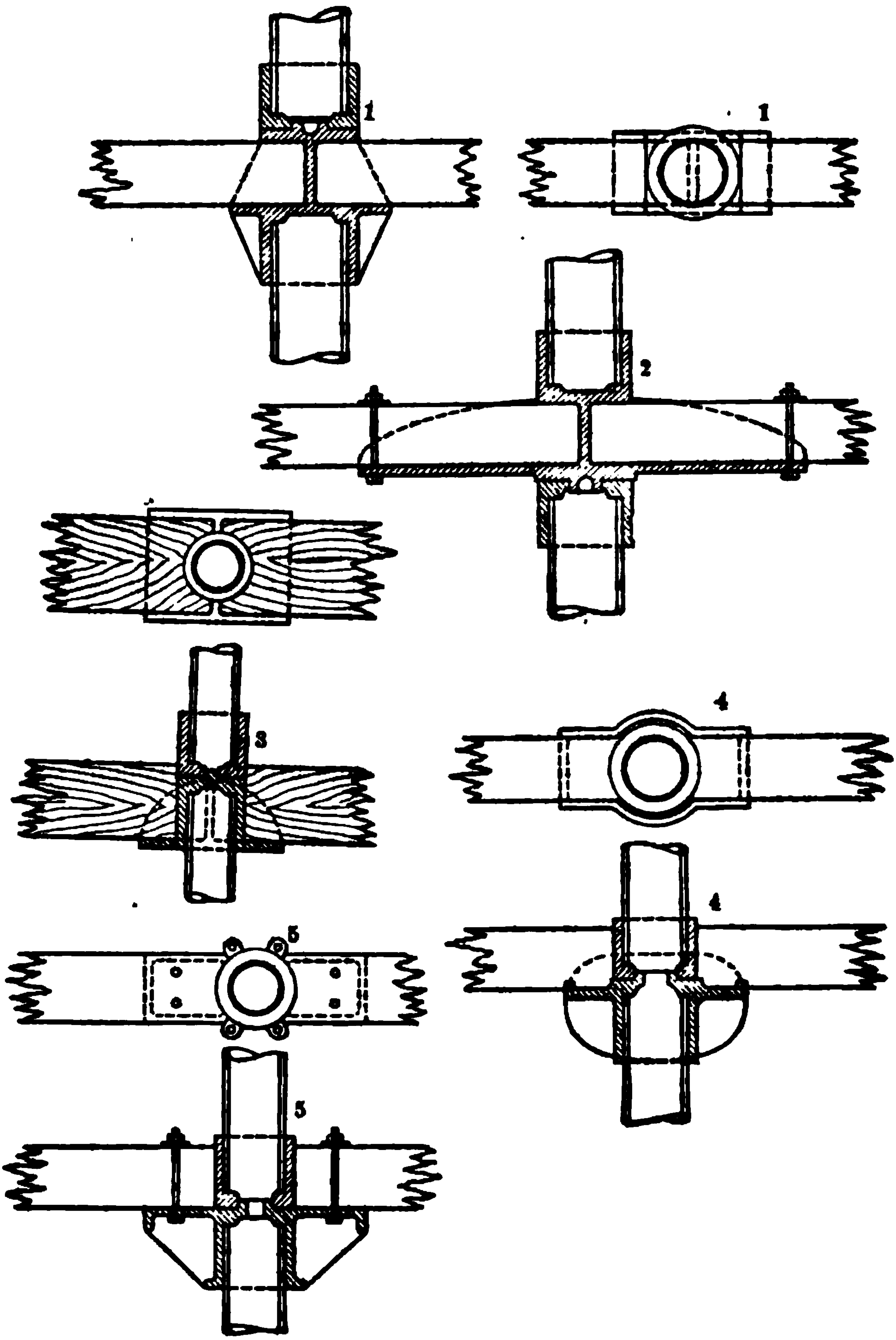


Fig. 10. Connections, Caps and Bases for Steel-pipe Columns

these steel-pipe columns filled with concrete are used, care should be t that the pipes are entirely filled, and that there are no air-spaces in the conc These concrete-filled columns, sometimes reinforced with smaller pipes,

a large carrying capacity. Pipe-columns may have their supporting power about doubled in many cases by concrete filling. (See, also, paragraph on Lally Columns, page 477). One type of steel post-cap used in connection with pipe.

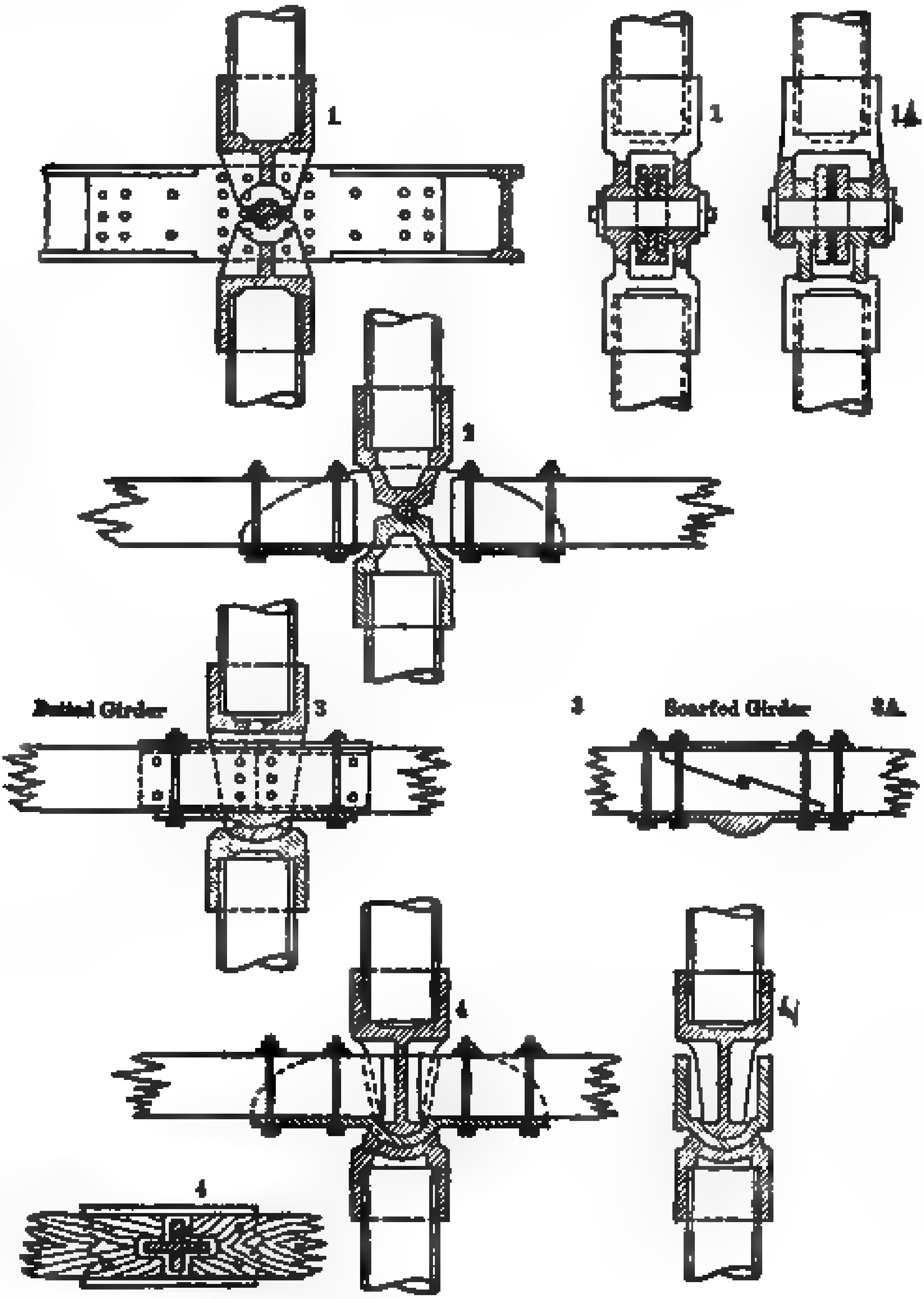


Fig. 11. Connections, Caps and Bases for Steel-pipe Columns

...ns to carry wooden girders is shown in Figs. 62 and 63 of Chapter XXII. ...e are many other forms of cast and wrought caps for pipe-columns. The ...n of proper caps and bases is the most difficult part of adapting tubular ...s to practical problems in building-construction. Figs. 10 and 11 show

various forms of steel-pipe column-connections, caps and bases sufficiently s
gestive to enable a designer to properly develop their details.

Advantages of Steel-Pipe Columns. A wrought-steel pipe when u
as a column generally has the following advantages:

(1) It will support a greater load per square inch of cross-section than
other shapes and styles of mild-steel columns of the same SLENDERNESS-RA
l/r, for most of the columns of different slenderness-ratios recently tested (1
and 1909) at the Watertown Arsenal.

(2) Its section has the greatest possible LEAST RADIUS OF GYRATION, r,
the same outside diameter and section-area. This makes pipe-columns es
cially advisable when it is desired to obstruct the view as little as possible, a
the corners of show-windows, in balcony-supports, etc.

(3) It may be used with greater slenderness-ratio, l/r, than any other sec
without reducing the load per square inch in order to conform to permiss
loading-rules, such as those of the New York City and the Chicago buil
codes.

(4) Its curved walls permit the use of relatively thinner material than
be used with columns with flat surfaces; that is, its thickness, t, divided by
outside diameter, d, may be $t/d = 1/80$ with as great security from WRINKL
called also BUCKLING, BULGING OR LOCAL FAILURE, as the box column, w
good practice of competent engineers limits to 1/40 of the unsupported wi
of flat surfaces. The ratio $t/d = 1/80 = 1/4''/20''$ is about the limit of practic
working of the ordinary lap-weld process, and all commercial pipes have a sma
ratio.

(5) Manufacturers are now regularly making pipes for sizes up to and inc
ing 16 in outside diameter, in lengths up to 40 ft.

Notes on the Use of Steel-Pipe Columns. The following general n
and suggestions should be observed in the use of steel pipe for columns:

(1) As in the case of columns of any construction, it is obvious that c
petent designing and detailing as well as proper fabrication of the END-CON
TIONS for pipe-columns be insisted upon. Otherwise the advantages of
circular section may be nullified.

(2) When the loading must be ECCENTRIC care must be exercised in the pr
selection and size of pipe to be used. The relative economy in the use of
circular section, however, increases with the length and slenderness of the colu

(3) A CAPITAL OR BASE should never be screwed to a pipe, because cut
the thread reduces the section. Where screw-threads must be used, only
area below the root of the threads should be considered as available for the
porting power.

(4) The ends of a pipe to be used for a column should always be FACED
in a lathe, the facing being normal to the general axis. A pipe should no
turned nor bored in fitting capitals or bases but, if possible, the capital or
should always be FORCED OR SHRUNK to an even bearing on the faced end o
pipe. Where the capital or base must be inserted, it is liable to start a wri
or buckle and the load should be adjusted to the probable lessening of suppo
power. The bearing surfaces in capitals and bases should be, of course, al
LATHE-FACED. It may be found that with careful foundry-work it is not n
sary to bore the castings; but it may, in some cases, be cheaper to use relati
poor foundry-work and bore the castings, as well as face the seats.

(5) PIN-ENDS OR BALL-AND-SOCKET ENDS are generally preferable to
or fixed ends for a slenderness-ratio l/r, of 100 or less, because tests show
columns so fitted usually carry heavier loads before failure. This is increas

...at as l/r decreases. Any form of end-connection of column that may ...a flexure from a falling floor may endanger the whole structure.

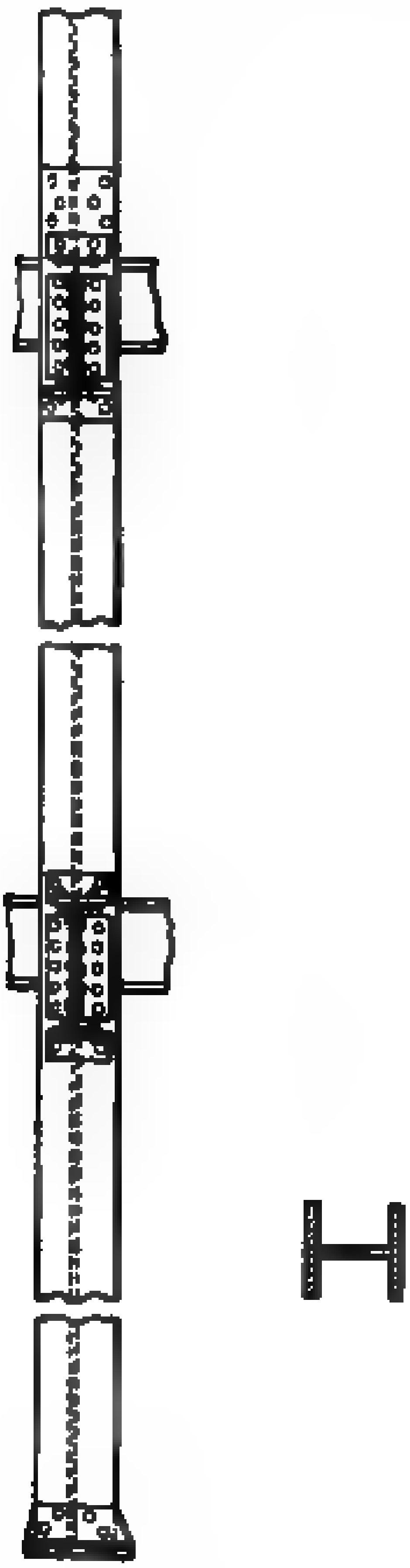


Fig. 12. Connections for Bethlehem H Columns

...) "All columns should have sufficient STIFFNESS to safely withstand the ...er deflecting forces to which they may be exposed. This usually involves ...iderations of eccentricity as well as of flexure due to transverse load.

...) "It is desirable to adhere always to the trade sizes of pipe known as ...EAST, STANDARD, EXTRA STRONG, DOUBLE-EXTRA STRONG, CASING, BOILER-

TUBES, etc., and avoid special production which usually entails delays and spec prices.

(8) Tables XII and XIII give the safe loads which STANDARD and EXTR STRONG steel-pipe columns are permitted to carry under the New York a Chicago codes. Philadelphia laws permit slightly grea loads. Supplementary tables of safe loads for DOUB EXTRA STRONG steel-pipe columns are furnished by t manufacturer and may be useful in cases where a mi mum diameter is required; but it should be remembe that such pipe always costs more per pound, owing to greater cost of manufacture.

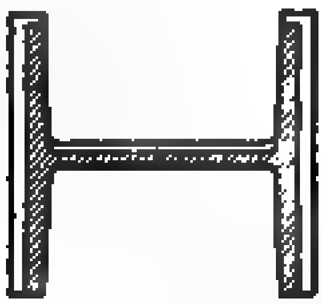


Fig. 13. Section of Bethlehem H Column Showing Variation in Area

H-Beam and I-Beam Struts and Columns. F struts and columns carrying light loads, H BEAMS a I BEAMS are probably the most economical, as the require very little riveting except for the splices an connections. Owing, however, to the narrow flanges o even the deepest I beams it is not practicable to rivet very heavy girders to them nor can they ordinarily be riveted to the web, because the latter is generally s

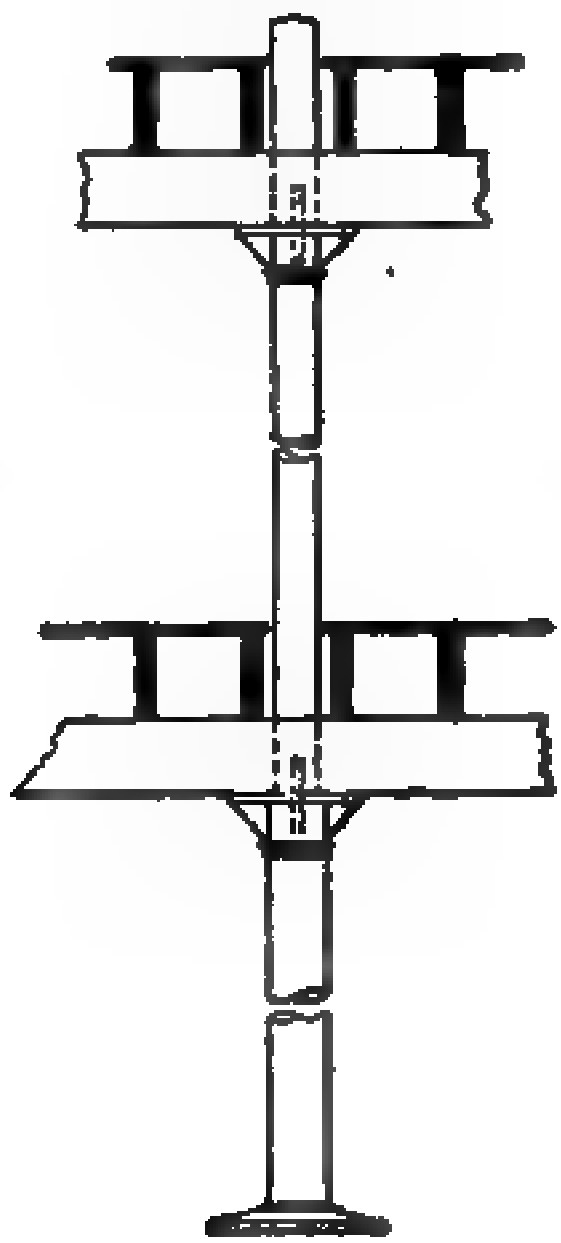


Fig. 14. Concrete-filled Lally Steel Column

Fig. 15. Lally Column. Typic Connections

thin that too many rivets will be required for the connection. Tables XVII a table of safe loads for the Carnegie steel H BEAMS or I BEAMS used columns.

Bethlehem Columns. As far as shop-work is concerned the Bethlehem columns are just as economical as the ordinary H-beam or I-beam columns as they, also, are rolled and not built up or assembled. The only fabrication required is that for the splice-plates and connections. Typical connections are shown in Fig. 12 from which the simplicity of detail and small amount of fabrication required are apparent. They are, moreover, superior to the I-beam columns because they afford a wider flange for attaching the beams and girders, besides being

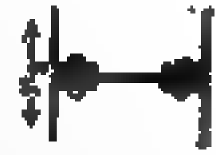


Fig. 16. Section of Steel Plate-and-angle Column

TYPICAL ANGLE-COLUMN

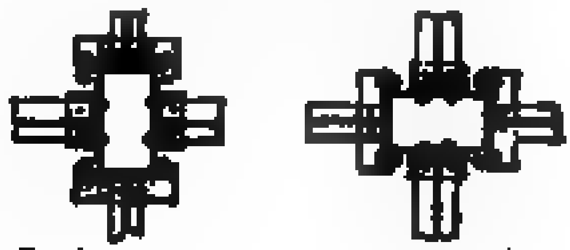
Bearing on masonry

Bearing on steel

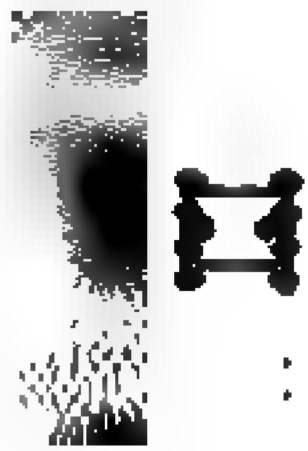
Fig. 17.* Connections for Steel Plate-and-angle Columns

*From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

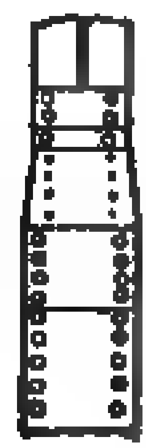
more economical of cross-section. Bethlehem columns are rolled in five size 6, 8, 10, 12, and 14 in in width, but by spreading the rolls, as shown i Fig. 13, the section-area of each width can be increased considerably. Th



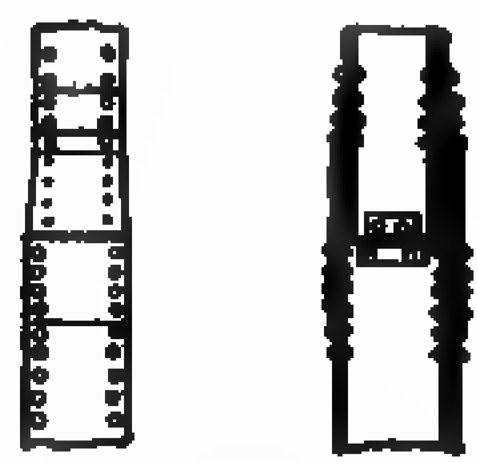
Section A-A Section B-B



TYPICAL SPLICE
Angle-column to Channel-column



TYPICAL SPLICE
Angle-columns, different sizes



TYPICAL CHANNEL-COLUMN
Bearing on steel

TYPICAL SPLICE
Channel-columns, different s

Fig. 16.* Connections for Steel Plate-and-channel Columns

section-areas of columns of the largest size may also be increased by riveting plates to the flanges. Tables of DIMENSIONS and PROPERTIES of Bethleh rolled steel columns and of the SAFE LOADS they will carry are given in Ta XVIII to XXI. Although these columns have been rolled in Germany

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

1902, it was not until the establishment in 1908 of the larger improved mills at Bethlehem, Pa., that these sections became available for use in this country. They are gradually superseding plate-and-angle and box columns, particularly those of the smaller sizes. The 6-in columns have been rolled since 1920.

Lally Columns. LALLY COLUMNS (see, also, pages 469 to 474 and Tables on page 516) are patented columns made with a circular steel shell, as shown in Fig. 14, and filled with a concrete composed of sand, cement and blue trap-rock, and thoroughly compressed. The larger columns have, in addition, a steel reinforcement, which makes a light, but strong support. They are in many buildings replacing masonry piers for supporting girders because of the saving in space, and are extensively used in mill-construction. Typical connections are shown in Fig. 15. The Lally formula for the safe loads in tons is given with Tables XXII and XXIII, page 516.

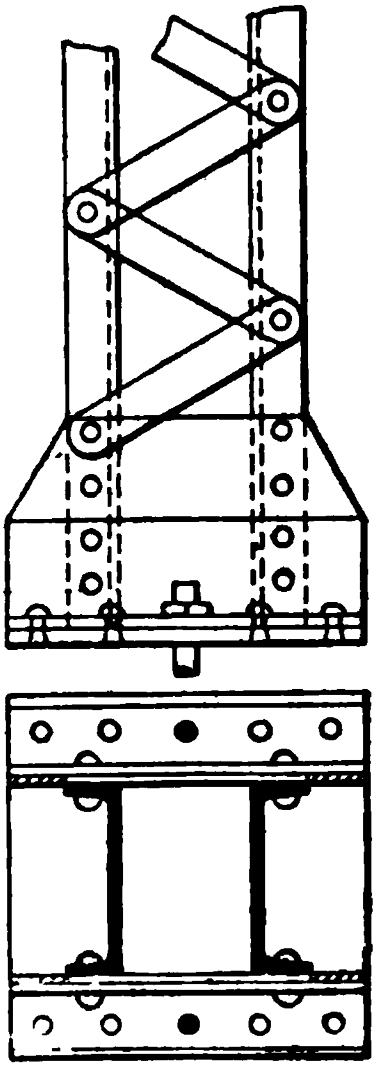


Fig. 19. Steel Channel-column with Lattice-bars

Plate-and-Angle Columns. Four angles and a plate riveted together as shown in Fig. 16 are now being extensively used in building-construction; particularly for columns having an unsupported length of less than 90 radii; also for the outer columns in steel-mill buildings, and for light columns supporting the roofs of railway stations, etc. Columns with this form of cross-section are especially convenient for making beam and girder-connections and for splicing, and are also well adapted to resist eccentric loads. The width of the plate is generally such that the LEAST RADIUS OF GYRATION is in the direction r_2, and this radius may be obtained directly from Tables XVI and XVII, pages 370 and 372.

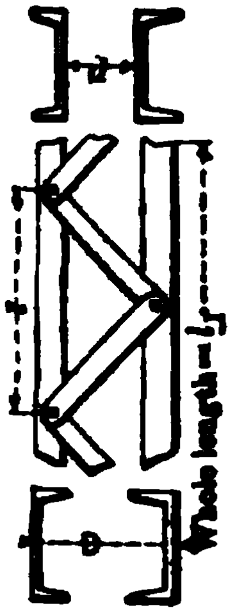


Fig. 20. Spacing of Lattice-bars in Channel-columns

Channel-Columns. Typical COLUMN-DETAILS for plate-and-angle and channel-columns, taken from the Carnegie Pocket Companion, 1915 edition, are shown in Figs. 17 and 18 and represent recent practice in office-building construction.

Lattice-Columns. Two channels, set back to back, at such a distance that the radii of gyration will be equal about both axes, and connected by lattice-bars, as shown in Fig. 19, make a very desirable column for moderate loads, as in the upper stories, or in buildings of three or four stories in height. For greater loads, short cover-plates may be riveted to the flanges in place of the lattice-bars. Such columns are very satisfactory, especially for making connections.

Rule for Latticing of Channels and Angles. When channels are connected by lattice-work, as in Fig. 20, in order that there may not be a tendency

in the channels to bend between the points of bracing, the distance l should be made equal to the total length of the strut multiplied by the least radius of gyration of a single channel, and the product divided by the least radius of gyration for the whole section; or,

$$l = rl_1/r_1$$

in which l = length between points of bracing;
l_1 = total length of strut;
r = least radius of gyration for a single channel;
r_1 = least radius of gyration for the whole section.

This same rule will also apply to angles, although with them the lattice-work is generally doubled, as in Fig. 21.

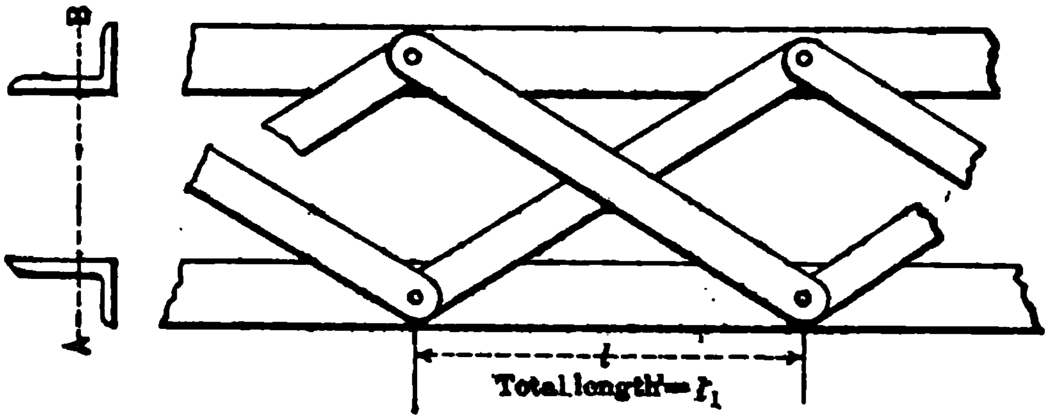


Fig. 21. Double Lattice-bars on Angle-columns

It is generally found desirable to make the distance l less than that obtained by the above formula. The inclination of the lattice-bars with the axis of the column or strut is usually about 60° for single and 45° for double bars.

The proper distance for d or D, Fig. 20, for a pair of channels, so that the radii of gyration will be the same in both directions, is given in Table VII, page 359.

The following tabulations are taken from the Handbook of the Cambria Steel Company, 1915 edition.

Sizes of Lattice-Bars to be Used with Latticed Channel-Columns

Depth of channels	Dimensions of lattice-bars		Weight of lattice-bars per foot	Center of hole to end of bar, a	Distance center to center of rivets, d		
	w	Thickness			Maximum		Minimum
in	in	in	lb	in	ft	in	in
6	1½	¼	1.28	1⅛	0	11½	6⅝
7	1¾	¼	1.49	1⅛	1	1½	7⅝
8	2	5/16	2.12	1¼	1	3	8 11/16
9	2	5/16	2.12	1¼	1	4½	9½
10	2	⅜	2.55	1¼	1	6½	10 11/16
12	2¼	⅜	2.87	1⅜	1	10½	13
15	2½	⅜	3.19	1½	2	2½	15 5/16

Sizes of Stay-Plates to be Used with Latticed Channel-Columns

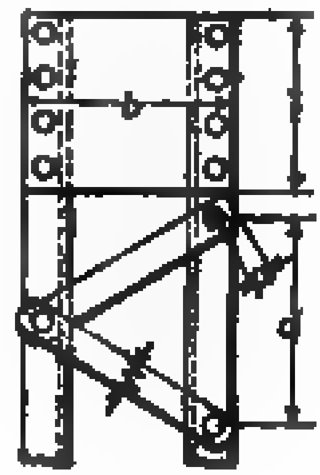
Minimum size of stay-plates at ends of columns			Weight of minimum stay-plates	Diameter of rivets	
b	Thickness	*l*			
in	in	in	lb	in	
8¼	¼	7½	4.38	⅝	
9¼	¼	10	6.55	⅝	
10½	5/16	9	8.37	¾	
11¼	5/16	12	11 95	¾	
12¼	⅜	12	15 62	¾	
14¼	⅜	15	22 73	¾	
16¼	⅜	15	25.90	¾	

Plate-and-Angle and Box Columns. Plate-and-angle columns, as shown in Fig. 16, requiring but two rows of rivets are very economical columns for buildings of moderate height, as they afford excellent opportunities for connecting the beams and girders. Tables of SAFE LOADS are given in Table XXIV of this chapter. When a more compact section is required than that afforded by the larger sizes, the section-area may be increased by riveting plates to the angles as shown in Fig. 22 which is a section of one of the columns in the Munic-

Fig. 23. Heavy Plate-and-angle Two-web Column

Fig. 24. Heavy Plate-and-angle Three-web Column

York City. This, however, greatly increases the expense nd it is therefore usually more economical to substitute s, or channel or box columns. For high buildings or heavy uired sectional areas of columns are greater than can be annel-columns or Bethlehem columns without flange-plates, f plates and angles, as shown in Fig. 23, which is one of the ers' Trust Company Building, New York City, will prob- more satisfactory. The thickness and number of web-plates be varied with the load to be supported. Ordinary con- umns are the same as those for CHANNEL-COLUMNS, shown tallest buildings and heaviest loads box columns with n in Fig. 24 are the best. They are used in the highest h as the Masonic Temple in Chicago, and the Bankers'

Trust Building, the Municipal Building, the Woolworth Building and the M
ropolitan Tower in New York City. Fig. 24 is a cross-section of one of
columns in the last-mentioned building. Details of a similar column use
the Bankers' Trust Company Building are shown in Fig. 7 on page 342.
is of course impracticable to give tables of SAFE LOADS for PLATE-AND-AN
COLUMNS with flange-plates and for BOX COLUMNS, owing to the great variet
combinations that can be used, but Example 10 of this chapter shows how
columns are designed and their strength determined. (See page 485.)

Steel Struts in Trusses. These are generally made of a pair of latt
channels, or of channels and plates for heavy trusses with pin-connections,
of either a pair of light channels or a pair of angles with uneven legs for l
trusses. For roof-trusses having a span not exceeding 80 ft, a pair of 4 by
3/8-in angles is generally sufficient for any of the compression-members un
they are subjected to TRANSVERSE STRESS; and the minor struts are very o
made of a pair of 3 1/2 by 2 1/2 by 1/4-in angles. The angles are placed from
3/8 in apart to permit the filler-plates used at the joints to go between th
For compression-members subject to transverse stress a pair of channels
erally offers the best section. If necessary the channels can be reinforced
plates at the top and bottom. A pair of angles, with a deep web-plate riv
between, is often used for the principles of Fink trusses where they are sub
to a slight transverse stress. (See, also, Fig. 6, page 1146.) For very l
compressive stresses and for short members a single angle is sometimes used;
this is not considered good practice, as it causes eccentric loading on the gu
plates at the truss-joints. A pair of small angles, or some other combina
with a symmetrical cross-section should always be used for truss-members
. Where angles are used in pairs they should be connected by a rivet and s
filler-plate or separator every two feet in length, to prevent them from sp
ing apart. In regard to the maximum length of steel struts in trusses
not considered good practice to use a strut whose unsupported length exc
150 times its least radius of gyration, or 50 times its least width.

13. Strength of Steel Columns. Formulas

Principles Governing the Resistance of Built-up Steel Columns.
fessor William H. Burr states * that "the general principles which gove
resistance of built-up columns may be summed up as follows: the ma
should be disposed as far as possible from the neutral axis of the cross-se
thereby increasing the radius of gyration, *r*; there should be no initial in
stress; the individual portions of the column should be so firmly secured t
other that no relative motion can take place, in order that the column m
as a whole, thus maintaining the original value of *r*." The experiments
by Professor Burr indicate that a closed column is stronger than an ope
It should also be remembered that any column such as an I beam, chan
angle, the cross-section of which has a maximum and a minimum rad
gyration, is not economical for use under a single concentric load, as the
mum radius of gyration must be used in the calculation, and part of the
rial is to a certain extent wasted when the ideal efficiency of the col
considered.

Formulas for Steel Columns. A great many FORMULAS are used
culating the strength of steel columns and struts, of the lengths usua
ployed in practice, but scarcely any two authorities agree upon the sa
These formulas may all be grouped into two general classes, those

* Elasticity and Resistance of the Materials of Engineering, by William H.

RANKINE'S FORMULA * (11) and those founded on the STRAIGHT-LINE FORMULA). (See the following paragraphs.) In the different formulas different values assigned to the ARBITRARY CONSTANTS. Previous to 1888 RANKINE'S or ...DON'S FORMULAS were almost universally used for all columns, although with ...or less variation in the constants employed. About 1885 Professor Burr, ...r having conducted a series of tests upon full-size column-sections deduced ...t is now known as the STRAIGHT-LINE FORMULA. As this is easier of applica...n than RANKINE'S FORMULA, it has gradually found favor with engineers, espe...lly as the results differ but little from those obtained by the older formula.

...mulas Compared. Which one, of all the formulas in use, should be ...ployed in calculating the safe load for columns is an open question, but the ...thor, after careful deliberation, has decided to recommend RANKINE'S FOR...MULA for the following reasons. In the first place it is safe and conservative ...d if it errs at all, it is on the side of safety; and in the second place it has a ...der application, as the values assigned to the arbitrary constants have been ...re generally agreed upon, whereas there is a greater variety in the values of ...e constants employed in the STRAIGHT-LINE FORMULA. Of course one is not ...e to choose when city laws compel the use of certain formulas. No tables of ...E LOADS for columns, satisfying the requirements of all cities, could be com...d. The author has accordingly thought it best to insert the various tables ...SAFE LOADS for different forms of columns as computed in the very latest ...dbooks although not necessarily based upon RANKINE'S FORMULA, and to ...t Table XI, specially computed and giving the comparative SAFE LOADS IN ...S PER SQUARE INCH OF METAL-AREA for columns, as determined by seven ...ferent formulas. (See pages 493 to 495.)

...mulas Used in Building Codes. RANKINE'S FORMULA (called GORDON'S ...ULA in many codes) is specified in the building codes of the following cities: ...adelphia, Pittsburgh, Baltimore, and Milwaukee; and in the Cambria ...book. The STRAIGHT-LINE FORMULA is specified in the building codes of ...w York City, Chicago, St. Louis, Minneapolis, Boston, and in many other ...s, and is used in the Carnegie and Bethlehem handbooks.

...mulas Used in Practice. The following formulas, in the opinion of the ...or, represent the best current practice. They are FORMULAS FOR SAFE ...s, S, in pounds per square inch of cross-section, on steel columns and struts. ...se formulas l is the LENGTH of the column in inches and r the LEAST RADIUS ...RATION of the cross-section. (See, also, Chapter X, pages 333, 344, etc.) ...e SAFE LOAD, P, for any column is equal to S, obtained by one of the follow...formulas, multiplied by the SECTION-AREA of the column in square inches;

$$P = AS \qquad (10)$$

...ine's formula, used in the Cambria handbook, is

$$S = \frac{12\,500}{1 + l^2/36\,000\,r^2} \qquad (11)$$

...formula recommended by Professor Burr is

$$S = 10\,000 - 40\,l/r \qquad (12)$$

...formula used by the American Bridge Company and Carnegie's Pocket ...panion is

$$S = 19\,000 - 100\,l/r \qquad (13)$$

...maximum of 13 000 lb per sq in.

* ...kine's formula is sometimes referred to as Gordon's formula, but Gordon used ...t lateral dimension or the diameter of the column instead of the least radius of ...of the cross-section.

The formula used by the American Railway Engineering Association the New York and Chicago building codes is

$$S = 16\,000 - 70l/r$$

with a maximum of 16 000 lb per sq in for New York and 14 000 for the ot

The formula used in the New York City building code previous to 1916

$$S = 15\,200 - 58\,l/r$$

The formulas used in the Catalogue of the Bethlehem Steel Company are

$$S = 16\,000 - 55\,l/r, \text{ for } l/r \text{ over } 55$$

and $S = 13\,000$ lb per sq in, for l/r under 55

Fowler's slightly modified formula for steel struts in trusses is

$$S = 12\,500 - 50\,l/r$$

The value 50 in Fowler's formula is 41⅔ when l is in inches, and 500 w is in feet.

For a comparison of most of these formulas, see Table XI, pages 493 to and the COMPARATIVE DIAGRAM OF FORMULAS, page 496.

14. Design of Steel Columns. Examples

Practical Use of Column-Formulas. Unlike the beam-formula the co formulas in general use do not give a direct method of calculating the dime of a column that will support a given load, owing to the presence in the co formula of two unknown quantities, A and r, which are dependent upo another. Hence in designing columns, the section must be first assume then tested for the safe load P, or for the maximum unit fiber-stress S. T an apparently roundabout method of designing columns, but unfortu there seems to be no more direct way. When a column is to be select designed, its axial load P is given and also its length and the condition ends. A proper allowable unit stress, S, is assumed, suitable for the material and for the conditions under which it is to be used, or in acco with the requirements of the local building code; or the value of S is gi the specification according to which the column is to be designed. A section is then selected in accordance with the principles explained on pag to 469. For this assumed cross-section A and r are determined and the stituted in the formula, which is solved for P. If the assumed dimensio a value for P that agrees with the actual load, they are correct. If, ho the resulting value of P is smaller than the actual load, the assumed size small, and it will be necessary to choose a larger size and solve again. the contrary, the actual load is less than the safe calculated load, a colum a smaller element of cross-section is assumed and a new value of P ob After a few trials a size that gives a satisfactory result for the required con will be found.

Examples Illustrating the Use of Column-Formulas and Tables. the column-tables in the last half of this chapter give the safe loads of the ity of column-sections of current practice, having determined which sec is most advisable to use under any given conditions, it is merely neces consult the tables and select the column of the required size to support the load.

Example 7. The following is an example showing the method of s BETHLEHEM ROLLED H COLUMNS for buildings.

Example Showing the Method of Selecting Bethlehem Rolled H Columns for Buildings

For illustration, the interior columns of an actual sixteen-story building are taken as example. The story-heights and the loads on the columns are given in the following tabulation:

Stories	Heights of stories, ft	Loads on columns, tons	Safe loads, tons	H column-section required: Dimensions: D, in	T, in	B, in	Weights of sections, lb per lin ft	Section-numbers
16th	12	27	55.0	7⅛	7/16	8.00	31.5	H8
15th	13	53	81.5	8⅜	11/16	8.12	48.0	H8
14th	14	79						
13th	13	104	132.2	10⅜	13/16	10.12	71.0	H10
12th	13	128						
11th	13	151	174.8	12¼	⅞	12.08	91.5	H12
10th	13	174						
9th	13	197	219.1	14¼	15/16	14.08	114.5	H14
8th	13	219						
7th	13	241	263.8	14⅝	1⅛	14.19	138.0	H14
6th	13	261						
5th	13	281	310.1	15	1 5/16	14.31	162.0	H14
4th	13	301						
3d	13	321	341.3	15¼	1 7/16	14.39	178.5	H14
2d	15	341						
1st	17	363	403.5	15¾	1 11/16	14.54	211.0	H14
Basement	12	395						

D is the depth of the column, *T* the thickness of the flanges and *B* the breadth of the flanges.

Columns for buildings are usually selected in lengths of two stories. By inspection of the tables of safe loads for H columns, it is found that no columns smaller than 14-in H sections have sufficient capacity for the lower stories. Where there is no limitation as to the size of the column, the column with the largest dimensions and having the required capacity will be the most economical. The unsupported length of a column should not exceed 150 radii of gyration, which is the limit of length for which safe loads are given in the tables. In the best practice the unsupported length of a column is frequently required not to exceed 120 or 125 times the least radius of gyration; various limits for l/r are indicated in the tables by zigzag lines. The safe loads given in the tables are for concentric or symmetric loading. When the loads are not centrally or symmetrically applied, the size of the column should be calculated by Formula (18), page 486.

Example 8. Suppose that in a 20-story office-building to be erected in Chicago, the load on each of the first-story columns, which are 16 feet in length, is 700 tons. What columns should be used?

Turning to Table XXI, page 515, giving ~~the safe loads for~~ Bethlehem 14-in columns it is seen that a 14-in 287.5-lb column, the heaviest rolled, will support only 549.3 tons; this type of column, therefore, cannot be used. More-

over a casual inspection of the tables of safe loads for plate-and-angle channel-columns shows that they are not suitable because of the thick flan and web-plates required. Consequently the columns in the lower stories probably have to be of the box type, with double or triple webs, as shown Figs. 23 and 24. The upper columns, however, may be of the plate-and-an or channel-type, whichever will be the more economical. The heaviest pl and-angle column, without flange-plates (Table XXIV, page 522), composed four 6 by 4 by ¾-in angles and one 12 by ½-in web, will support, for a len of 14 feet, the height of most of the upper stories, 469 000 lb; and a channel-umn (Table XXVI, page 541) composed of two 12-in 30-lb channels and 14 by ¾-in plates will support 502 000 lb. The former weighs 125 and the la 131.4 lb per lin ft, so there is not much choice as far as economy of materia concerned. The channel-column, however, requires four rows of rivets w the plate-and-angle column requires only two rows, so this added expens fabrication would have to be considered. Assuming, however, that the pl and-angle type is more desirable, the next step is to design the individ columns.

The load upon each of the uppermost columns, which are 20 ft in length 70 000 lb. Turning to Table XXIV, page 518, it will be seen that a colu composed of four 4 by 3 by ⅜-in angles and one 8 by ⅜-in web will supp for a length of 20 ft, 77 000 lb; but this load is below the lower zigzag line hence the slenderness-ratio of the column exceeds 120. Assuming, for the p pose of illustration, that the limit of l/r is 120, a heavier section must selected. On page 519 of Table XXIV, continued, it is seen that the ligh 20-ft column, for which l/r does not exceed the required ratio, is one compo of four 5 by 3½ by ⅜-in angles and one 10 by ⅜-in web, and that for a len of 20 ft it will support 121 000 lb, or 51 000 lb more than will come upon it.

Continuing the design of the columns, suppose that one in the 14th st 14 ft in length, supports 175 tons, or 350 000 lb. From Table XXIV, page it is found that a column composed of four 6 by 4 by ⅝-in angles and one 12 ½-in web-plate, for a length of 14 ft, will carry 373 000 lb. In the table, safe load is calculated by Formula (13), whereas the Chicago Building C specifies Formula (14). Hence, as this building is to be erected in Chicago, chosen column must be tested by the latter formula. Its A is 29.44 sq in its least r, 2.65 in. To test it by the formula, $l = 14$ ft $\times 12 = 168$ in, and $l/$ 168 in/2.65 in $= 63$. Substituting in Formula (14), $S = 16\,000 - (70 \times 63$ $16\,000 - 4\,410 = 11\,590$ lb per sq in. From Formula (10), the safe load for column, $P = AS = 29.44$ sq in $\times$ 11 590 lb per sq in $=$ 341 209 lb, which is than the actual load. Therefore, the next heavier column, with angles 1½ thick, should be selected.

Example 9. In an office-building to be erected in Philadelphia, the use of Bethlehem rolled-steel H columns has been decided upon. One of these umns, 15 ft in length, supports 170 000 lb, or 85 tons. What should be the of this column?

According to Table XIX, page 508, giving the safe loads for Bethlehem umns, a 10-in 49-lb column, 15 ft in length, will carry 86.3 tons, an appare safe load. Bethlehem-column loads, however, are calculated by the straight-formula, whereas in Philadelphia, Rankine's (called Gordon's) formula is standard. This formula with the arbitrary constants inserted is

$$S = \frac{16\,250}{1 + \frac{1}{11\,000}(l/r)^2}. \quad \text{(See Table XI, page 493.)}$$

from Table XIX, $A = 14.37$ sq in and the least $r = 2.49$ in; l is 15 ft or 180 in
$l/r = 180 \text{ in}/2.49 \text{ in} = 72.3$.

Substituting in the formula,

$$S = \frac{16\,250}{1 + \frac{1}{11\,000}(72.3)^2} = \frac{16\,250}{1 + 5\,227/11\,000} = \frac{16\,250}{16\,227/11\,000}$$

$$= \frac{16\,250 \times 11\,000}{16\,227} = \frac{178\,750\,000}{16\,227} = 11\,015 \text{ lb per sq in}$$

and from Formula (10), page 481,

$$P = AS = 14.37 \text{ sq in} \times 11\,015 \text{ lb per sq in} = 158\,285 \text{ lb or } 79.1 \text{ tons},$$

which is less than the tabular load. Hence the next heavier column, weighing 54 lb per sq ft, would have to be used.

Example 10. Figure 7, page 342, shows the cross-section of one of the basement-columns in the Bankers' Trust Company's Building, New York City. It is 20 ft in length and supports 2 230 tons. Is the column safe?

The first step is to find its least radius of gyration which is equal to $\sqrt{I/A}$. The least moment of inertia of this section was found to be 17 030. (See page 343.) The area is made up as follows:

FLANGES. The flanges are composed of six 27 by ¾-in plates and two 27 by 1 1/16-in plates. The area of the cross-section of each 27 by ¾-in plate is 20.25 sq in and of the six plates, 121.50 sq in. The area of the section of each 27 by 1 1/16-in plate is 18.56 sq in and of the two plates, 37.12 sq in. Hence the total sectional flange-area is 121.50 + 37.12 =	158.62 sq in
FLANGE-ANGLES. Each flange-angle is 6 by 6 by 15/16 in. Its section-area is 10.38 sq in. Hence for the four, $A = 10.38 \times 4 =$	41.52 sq in
OUTER WEB. The outer web-plates are each 18 by 1 1/16 in. The area of each one is 12.375 sq in and of the eight	99.00 sq in
WEB. Each web-angle is 6 by 3½ by 15/16 in with a section-area of 8.03 sq in; and for four angles the section-area is	32.12 sq in
WEB. The web is composed of two 18 by 9/16-in plates, each with a section-area of 10.125 sq in. For two the area is	20.25 sq in
The area of the entire section, therefore, is	351.51 sq in

$$r^2 = I/A = 17\,030/351.5 = 48.5 \quad \text{and} \quad r = \sqrt{48.5} = 7 \text{ in}$$

$$l = 20 \text{ ft} = 240 \text{ in and } l/r = 240 \text{ in}/7 \text{ in} = 34.3$$

Substituting in the former New York City building code Formula (15), page 482,

$$S = 15\,200 - 58 \times 34.3 = 15\,200 - 1\,989 = 13\,211 \text{ lb per sq in}$$

From Formula (10)

$$P = AS = 351.5 \text{ sq in} \times 13\,211 \text{ lb per sq in} = 4\,643\,666 \text{ lb, or } 2\,321 \text{ tons.}$$

Hence the column is perfectly safe.

15. Eccentric Loading of Steel Columns

General Principles. Where columns are used in tiers, one above another, the beams and girders which they support must necessarily rest upon brackets projecting or extending varying distances beyond the shell or section-areas or of the columns. Such connections cause BENDING MOMENTS in the columns. When equal loads are applied at equal distances on opposite sides of a column,

the bending moments caused by them in the column balance each other, an the CENTER OF STRESS may be considered as coinciding with the axis of th column. When, however, a load is applied on one side (Fig. 25) without corresponding load on the opposite side, it is called an ECCENTRIC LOAD and th area of the cross-section of the column should be increased correspondingl There is unfortunately no direct method by which this additional area can l determined. The usual method of procedure is to assume a section in exce of that required to support the total load and then compute the fiber-stress d to the combined balanced and eccentric loads. If this works out too lar or too small another trial is made.

Formula for Eccentric Loads on Steel Columns. The following form (compare with Fig. 25) is used to determi the combined fiber-stresses due to the conce tric and eccentric loads (See, also, page 453):

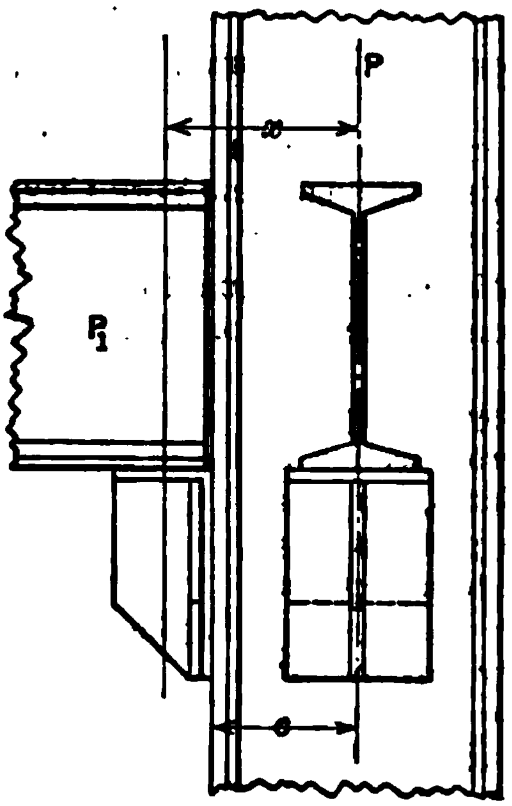


Fig. 25. Channel-column with Eccentric Load. Elevation

Let P = the concentric or balanced load pounds,
P_1 = the eccentric load in pounds,
M = the bending moment due to t eccentric load in inch-pounds = P
x = the eccentricity of the load P_1 inches. (See note below:)
I = the moment of inertia of the ar of the cross-section of the colu about an axis at right-angles to t direction of the bending,
c = the distance of the outermost fiber the cross-section from the same a
A = the area of column-section in squ inches and
S = the actual fiber-stress in pounds p square inch

Then

$$S = (P + P_1)/A + Mc/I \qquad ($$

Note. In measuring the ECCENTRICITY, the distance, x, is generally measu from the axis of the column to the center line or half-breadth line of the brac or bearing.

Examples of Eccentric Loading of Steel Columns. The following e amples illustrate the use of the formula and tables in determining the safe eccentric loads for steel columns.

Example 11. The total load on the top of a column 32 ft in length is 194 000 lb, of which 30 000 lb come from the end of a girder. There is no corresponding load on the opposite side. (See Fig. 26.) It is proposed to use a channel-column. What is the size of the required column?

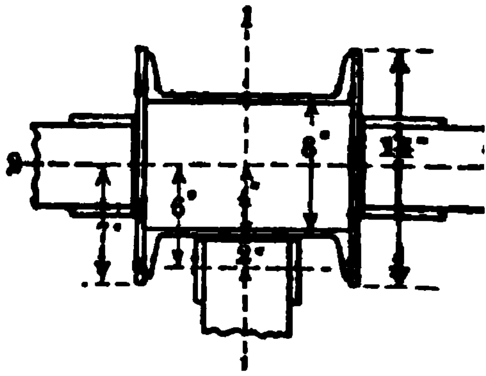


Fig. 26. Channel-column w Eccentric Load. Section

By referring to Table XXVI, page 539, it is seen that a column composed of two 12-in 20.7-lb channels and two 14 by ⅜-in plates will support, for a length of ft, 227 000 lb, a somewhat greater load than will come on the column. I the section of this column, I_{2-2} = 415, A = 22.56 sq in, r = 4.29 in and $L/$

$84/4.29 = 89$. Substituting in Formula (11), page 481, to find the safe unit fiber-stress

$$S = \frac{12\,500}{1 + \frac{1}{36\,000}(l/r)^2} = \frac{12\,500}{1+(89)^2/36\,000} = \frac{12\,500}{1+7\,921/36\,000} = \frac{12\,500}{43\,921/36\,000}$$

$$= \frac{12\,500 \times 36\,000}{43\,921} = \frac{450\,000\,000}{43\,921} = 10\,245 \text{ lb per sq in}$$

The actual stress in pounds per square inch of the column-section is found by Formula (18), $S = (P + P_1)/A + Mc/I$. $P = 164\,000$ lb, $P_1 = 30\,000$ lb, $A = 22.56$ sq in and $M = P_1x$ in-lb. $x =$ the distance in inches from the axis 2-2 of the column to the outside of the web, plus the distance from the outside of the web to the center of the bracket. The former distance can be found from Table XXVI. It is 4 in. Let the distance from the outside of the web of the channel to the center of the bracket riveted to the web of the channel be 2 in, the projection of the bracket being 4 in; then x, the lever-arm of the moment of the load P_1, or the eccentricity, is 4 in + 2 in = 6 in. M, therefore, is P_1x or 30 000 lb × 6 in. c is 7 in, since the plates are 14 in wide. $I_{2-2} = 415$. Substituting in Formula (18)

$$S = \frac{164\,000 + 30\,000}{22.56} + \frac{30\,000 \times 6 \times 7}{415} = 8\,600 + 3\,036 = 11\,636 \text{ lb per sq in}$$

As this exceeds the safe unit fiber-stress of 10 245 lb per sq in, the column-section is too small.

For a second trial, consider a 12-in, 20.7-lb channel-column with 14 by ½-in plates. For this section, $I_{2-2} = 473$, $A = 26.06$ sq in, $r_{2-2} = 4.26$ in and $l/r = 384/4.26 = 90$.

$$S = \frac{12\,500}{1+(90)^2/36\,000} = \frac{12\,500}{1+8\,100/36\,000} = \frac{12\,500}{44\,100/36\,000}$$

$$= \frac{12\,500 \times 36\,000}{44\,100} = 10\,204 \text{ lb per sq in.}$$

The actual stress from Formula (18), as before, is

$$S = \frac{164\,000 + 30\,000}{26.06} + \frac{30\,000 \times 6 \times 7}{473} = 7\,444 + 2\,664 = 10\,108 \text{ lb per sq in}$$

As this is less than the safe stress of 10 204 lb, the second selection is safe.

Example 12. A Bethlehem H column 14 ft long carries 90.56 tons, of which 15.52 tons are eccentric, being applied to the flange of the column as shown in Fig. 27, the distance from the outside of the flange to the center of the bearing being 2 in. What is the size of the column required?

Try a 12-in, 84.5-lb column, which, for a length of 14 ft, or 168 in, will carry 161.4 tons (Table XX). For this column, $A = 24.92$, $r_{2-2} = 3.03$, $I_{1-1} = 676.1$, $I_{2-2} = 228.5$ and l is 14 ft, or 168 in; hence $l/r = 168/3.03 = 55$. Substituting in Formula (15), assuming that that formula is specified, $S = 15\,200 - 58 \times 55 = 12\,010$ lb per sq in. Since the eccentric load causes bending in a direction at right-angles to the axis 1-1, Fig. 27, the bending moment due to the eccentric load is P_1, or 15.52 tons or 31 040 lb, multiplied by its lever arm x, which is the distance from the axis 1-1 to the outside of the flange plus the distance from this surface to the center of bearing. The former dimension, taken from the Bethlehem Catalogue, is 6⅛ in and the latter is 2 in; hence $x = 8⅛$ in or for convenience, 8 in. The distance, c, also, of the outermost fiber from the axis

1–1 is 6¹⁄₁₆ in, which for convenience will be considered 6 in. I_{1-1} about th axis 1–1 is 676.1. Substituting in Formula (18), $S = (150\,080 + 31\,040)/24.92 +$ $(31\,040 \times 8 \times 6)/676.1 = 7\,268 + 2\,204 = 9\,47$ lb per sq in. As this is far below the sa stress of 12 010 lb, the column selected too large, and a smaller one, probably a 12-i 64.5-lb column would prove sufficient.

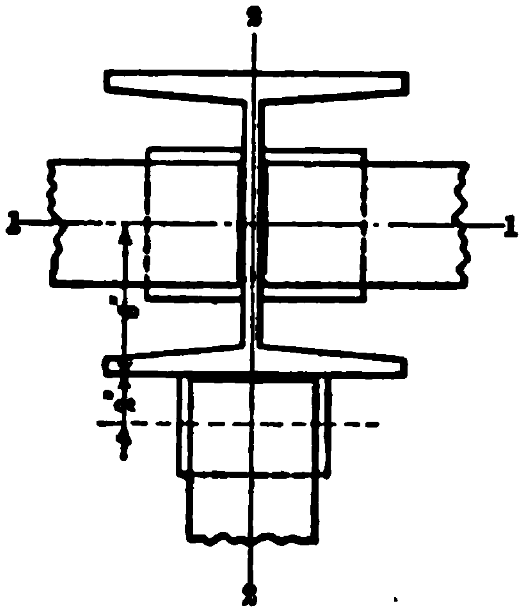


Fig. 27. Bethlehem H Column with Eccentric Load

Suppose, on the other hand, the eccentr load were applied to the web and the balanc loads to the flanges. The safe unit fibe stress, as before, is 12 010 lb per sq in, fo no matter how the loads are applied, the sa unit stress determined by reference to t least radius of gyration, r_{2-2}, should not b exceeded. Under the second condition loading the eccentric load, also, will cau bending about this same axis 2–2; hence Formula (18) the I for this axis, which 228.5, must be used. $x = 2$ in + 0.25 in 2.25 in, 0.25 in being one-half the thickne of the web. (See the Bethlehem Catalogue.) Hence, from Formula (18), t actual unit fiber-stress is $S = (150080 + 31\,040)/24.92 + (31\,040 \times 6 \times 2.25)/228$ $= 7\,268 + 1\,835 = 9\,103$ lb per sq in.

16. Tables of Safe Loads for Steel Columns

Safe Loads per Square Inch of Metal-Area for Steel Columns a Struts. To lessen the labor of calculating the strength of steel columns a struts, of whatever shape, the author has computed Table XI, which gives SA VALUES of S for ratios of l/r varying from 30 to 120. For ratios of l/r which a not whole numbers, the values can be readily interpolated. The values in t table should correspond exactly with the results obtained by using the corr sponding formulas.

Safe Loads for Steel-Pipe Columns. Tables XII and XIII give the SA LOADS for STEEL-PIPE COLUMNS. These loads are based upon the formula reco mended by the New York and Chicago Codes, $S = 16\,000 - 70\,l/r$. (See Ste Pipe Columns, pages 469 to 474.)

Safe Loads for Channel and Angle-Struts. Tables XIV, XV and X give the SAFE LOADS for standard CHANNELS and ANGLES used as STRUTS. On those sizes that are most commonly used are given. In Table XIV the SA LOADS for both the minimum and the maximum RADIUS OF GYRATION are give If the strut is used also as a beam, or is stayed so that it cannot bend sidewi the larger value may be taken; but if free to bend in either direction, then t smaller value should be taken. If the struts are subjected to a TRANSVER STRESS they should be computed as explained under the heading Strut-Bean pages 571 and 572.

Safe Loads for Steel-Beam Columns, Bethlehem Columns, Lally C umns, Plate-and-Angle and Channel Columns. Tables XVII to XXV giving the SAFE LOADS for these columns, were not computed by the auth but by the different manufacturers; they are, however, believed to be perfec safe, provided that an increase in area is made for ECCENTRIC LOADS.

Use of Table XI for Determining Safe Loads for Steel Column This table will be found of great assistance in calculating the strength of c

sions and of struts and also in making calculations for eccentric loads. To use it to find the strength of a column, it is merely necessary to multiply the value corresponding to the SLENDERNESS-RATIO of the column, by the SECTION-AREA, the result being the SAFE LOAD the column can support. As an illustration of this, the column considered in Example 8 has a slenderness-ratio of 63 and a section-area of 29.44 sq in. Its strength is to be calculated by the Chicago Building Code formula, the results of which are tabulated in the sixth column of Table XI. From this the value of a slenderness-ratio of 63 is 11 590 lb per sq in. Therefore, by the rule stated above, the safe load is 11 590 lb per sq in × 29.44 sq in = 341 209 lb. In Example 10, the column in the Bankers' Trust Company Building has a slenderness-ratio of 34.3, and an area of 351.5 sq in. The value corresponding to 34, from column 5 of Table XI, is 13 228 and for 35 it is 13 170 lb per sq in; hence for 34.3 it would be about 13 211 lb per sq in. Accordingly, the safe load is 13 211 lb per sq in × 351.5 sq in = 4 643 666 lb. Column 5 of Table XI gives values for old New York code.

Example 13. What is the safe resistance of a strut composed of two 5-in 9-lb channels, separated ¾ in and free to bend in either direction, the length of the strut being 7 ft 6 in?

Solution. From Table XVIII, page 374, the least radius of gyration for this section is 1, hence $l/r = 90/1 = 90$. From the eighth column of Table XI, the value of S opposite 90 is 8 000 lb per sq in; the safe load, then, is equal to 8 000 lb per sq in, multiplied by the area of the two channels, 5.26 sq. in, or 42 080 lb.

Example 14. What is the safe stress for a 7-in 15.5-lb I beam when used as a strut? It is 90 in in length and free to bend in either direction.

Solution. From Table IV, page 355, the least radius of gyration of this section is 0.78, and the area is 4.43 sq in. $l/r = 90/0.78 = 115.4$. From the eighth column of Table XI, the value opposite 115 is 6 750 and opposite 116 it is 6 700 lb per sq in; so for 115.4 it would be about 6 730 lb per sq in. The safe load, therefore, is 6 730 lb per sq in × 4.43 sq in = 29 814 lb.

By means of the tables and rules given in Chapter X the SECTION-AREA and LEAST RADIUS OF GYRATION of any standard section or any combination of sections may be found; and once these are determined the strength of a strut or column may be readily computed, as in the above examples.

Use of Table XI for Eccentric Loads for Steel Struts. As an illustration of its application to determine eccentric loads, refer again to Example 11. The value of l/r for this column is 89. The safe unit fiber-stress was found to be, by Formula (11), 10 245 lb per sq in. The practically identical result can be obtained by looking for the value opposite 89 in column 2 of Table XI. It is found to be 10 250 lb.

Proportion of Floor-Loads Borne by Columns. (See, also, pages 148 to 152.) In tall buildings it is customary to reduce the COLUMN-LOADS somewhat from the loads used in calculating the floor-beams. This is done on the theory that it is quite impossible for the entire floor-area of every story to be loaded to the maximum limit at the same time. For all buildings except warehouses it would seem, in general, to be good practice to design the columns to carry all the DEAD LOAD and 75% of the assumed LIVE LOAD. Of course city laws vary in these requirements. Thus, if in an office-building, the dead load, or weight of the floor-construction, is 80, and the live load 80 lb per sq ft, the load on the columns would be 80 + 60 = 140 lb per sq ft times the floor-area supported by the column. In some cases the reduction might be even greater, depending upon the live load assumed and the position of the column in the building, the reductions being greater in the lower than in the upper stories.

The Building Code of New York City specifies that for buildings exceedi five stories in height the COLUMN-LOADS shall be made up as follows: For t roof and top floor the full live loads shall be used; for each succeeding low floor it shall be permissible to reduce the live load by 5% until 50% of the li load is reached, when such reduced loads shall be used for all remaining floo (For assumed loads for office-buildings, required by the building codes of se eral cities, see page 151).

Column-Sheets. In a high building the COLUMN-LOADS vary to such extent and are made up of so many elements, that to avoid omissions and err it is necessary to make a TABULATED LIST of all the loads transferred throu the columns to the footings. In a building of skeleton construction the COLUM LOADS include floor and roof-loads, wind-loads, spandrel and pier-loads, t weight of the columns themselves and their fire-proof covering, and in some ca special loads, such as tanks, vaults, safes and elevator-loads. In tabulati the FLOOR-LOADS it is advisable to separate the dead and live loads for c venience in proportioning the footings. (See, also, pages 148 to 160.) Formu for computing the WIND-LOADS on columns are given in Chapter XXIX; th loads, also, are considered as live loads. ECCENTRIC LOADS should always tabulated separately from the balanced column-loads. On page 491 is sho a form of COLUMN-SHEET which combines all ordinary requirements. T TOTAL LOAD for each story is the sum of all of the loads above. The SCHEDU on page 492 shows a very convenient form for column-lengths and column-pa

Important Notes Regarding Safe Loads on Columns. (See pages 504 a 505, and 517 to 554.) "For ratios of l/r up to 120 and for greater ratios up 200, use the values given in the following table for the allowable stress pounds per square inch. For intermediate ratios, use proportional amounts.

Ratio	Amount	Ratio	Amount
60	13 000	130	6 500
70	12 000	140	6 000
80	11 000	150	5 500
90	10 000	160	5 000
100	9 000	170	4 500
110	8 000	180	4 000
120	7 000	190	3 500

"(5). For bracing and combined stresses due to wind and other loadi the permissible working stresses may be increased 25 per cent, provided section thus found is not less than that required by the dead and live lo alone.*

"(6). General. The effective or unsupported length of main compressi members shall not exceed 120 times, and for secondary members 200 times, least radius of gyration."*

The values for ratios of l/r above 120 are computed from the formula

$$S = 13\,000 - 50\, l/r,$$

but the important condition should be observed, that for l/r above 120, co pression-members should never be used for main members but only as second members subject to wind-stresses, etc. (See, also, page 495 for maximum ra of l/r for main members and for secondary members, such as bracing struts, et

* From the Construction Specifications of the American Bridge Company.

Form of Column-Sheet

Story	Character of loading	Column No. 1: Load on column, concentric	Column No. 1: Load on column, eccentric	Column 2
18th top	Roof and ceiling, dead load......			
	Roof and ceiling, live load.......			
	Masonry piers..................			
	Spandrels, cornice, etc...........			
	Elevators......................			
	Tanks.........................			
	Column and casing..............			
	Wind-load......................			
	Total......................			
	Sectional area required..........	sq in	sq in	
17th	From column above.*			
	Floor, dead load................			
	Floor, live load.................			
	Masonry piers..................			
	Spandrels......................			
	Safes, vaults, etc................			
	Column and casing..............			
	Wind-load......................			
	Total......................			
	Sectional area required..........	sq in	sq in	
Basement	From column above.*			
	Floor, dead load................			
	Floor, live load.................			
	Masonry piers..................			
	Spandrels......................			
	Sidewalk.......................			
	Column and casing..............			
	Wind-load......................			
	Total......................			
	Sectional area required..........	sq in	sq in	
	Deduct (½) live load...........			
Footings	Total footing-load...........			
	Area of footing required.........	sq ft		

* In bringing down the load from the column above, the eccentric loads may be added …centric loads and their sum placed in the first column.

Schedule of Column-Lengths and Parts

	Column No. 1		Column No. 2	
Roof-line				
Top of columns	1' 6½"			
7th story	½"	Four angles 4"×3"×5/16" One plate 7"×5/16"		
7th Floor-line	23' 4"			
6th Story				
6th Floor-line	2' 2"	Four angles 5"×3"×3/8" One plate 7"×3/8"		
5th Story	13' 10"			
	¾"			
5th Floor-line	4¼"			
	¾"			
1st Floor-line	1' 2¼"	Four Z's 4"×3/8" One plate 7"×7/8"		
Basement	11' 2"			
Top of stool				
Grade 15.0	8¼"			

III. Safe Loads in Pounds per Square Inch of Metal Area for Steel Columns and Struts

l = length in inches r = least radius of gyration in inches

ankine's (Gordon's) and Cambria	Philadelphia	Boston,* before 1919	Washington, D. C.†	Chicago ‡ and N. Y.	Am. Bridge Co. and Carnegie	Fowler's formula for struts	
$\dfrac{12\,500}{1+\dfrac{l^2}{36\,000r^2}}$	$\dfrac{16\,250}{1+\dfrac{l^2}{11\,000r^2}}$	$\dfrac{16\,000}{1+\dfrac{l^2}{20\,000r^2}}$	$15\,200-58\,l/r$	$16\,000-70\,l/r$ Chi. 14 000 max N. Y 16 000 max	$19\,000-100\,l/r$ 13 000 max	$12\,500-$ [illegible] l/r	l/r
II	III	IV	V	VI	VII	VIII	IX
12 195	15 000	15 310	13 460	13 900	13 000	11 000	30
12 170	14 945	15 265	13 402	13 830	13 000	10 950	31
12 155	14 865	15 220	13 344	13 760	13 000	10 900	32
12 135	14 785	15 175	13 286	13 690	13 000	10 850	33
12 110	14 705	15 125	13 228	13 620	13 000	10 800	34
12 090	14 620	15 075	13 170	13 550	13 000	10 750	35
12 065	14 535	15 025	13 112	13 480	13 000	10 700	36
12 045	14 450	14 975	13 054	13 410	13 000	10 650	37
12 020	14 365	14 925	12 996	13 340	13 000	10 600	38
11 995	14 275	14 870	12 938	13 270	13 000	10 550	39
11 970	14 185	14 815	12 880	13 200	13 000	10 500	40
11 945	14 095	14 760	12 822	13 130	13 000	10 450	41
11 920	14 005	14 705	12 764	13 060	13 000	10 400	42
11 890	13 915	14 650	12 706	12 990	13 000	10 350	43
11 860	13 820	14 590	12 648	12 920	13 000	10 300	44
11 835	13 725	14 530	12 590	12 850	13 000	10 250	45
11 805	13 630	14 470	12 532	12 780	13 000	10 200	[illegible]
11 780	13 535	14 410	12 474	12 710	13 000	10 150	47
11 750	13 440	14 350	12 416	12 640	13 000	10 100	48
11 720	13 340	14 285	12 358	12 570	13 000	10 050	49
11 690	13 240	14 220	12 300	12 500	13 000	10 000	50
11 660	13 145	14 160	12 242	12 430	13 000	9 950	51
11 620	13 045	14 095	12 184	12 360	13 000	9 900	52
11 595	12 945	14 030	12 126	12 290	13 000	9 850	53
11 565	12 845	13 965	12 068	12 220	13 000	9 800	54
11 530	12 745	13 900	12 010	12 150	13 000	9 750	55
11 500	12 645	13 835	11 952	12 080	13 000	9 700	56
11 465	12 545	13 770	11 894	12 010	13 000	9 650	57
11 430	12 445	13 700	11 836	11 940	13 000	9 600	58
11 400	12 345	13 630	11 778	11 870	13 000	9 550	59
11 365	12 240	13 560	11 720	11 800	13 000	9 500	60
11 330	12 140	13 490	11 662	11 730	12 900	9 450	61
11 295	12 040	13 420	11 604	11 660	12 800	9 400	[illegible]
11 260	11 940	13 350	11 546	11 590	12 700	9 350	63
11 225	11 840	13 280	11 488	11 520	12 600	9 300	[illegible]
11 185	11 740	13 210	11 430	11 450	12 500	9 250	65
11 150	11 640	13 140	11 372	11 380	12 400	9 200	66
11 115	11 540	13 070	11 314	11 310	12 300	9 150	67
11 080	11 440	13 000	11 256	11 240	12 200	9 100	68
11 040	11 340	12 925	11 198	11 170	12 100	9 050	69

ton Code, 20 000—100 l/r; maximum, l/r, 160; maximum stress, 12 000.
New York Code, Newark, N. J., Atlanta, Ga., Worcester, Mass., etc.
Engrg. Ass'n the same up to l/r, 100, for main members. Maximum, 14 000.

Table XI (Continued). Safe Loads in Pounds per Square Inch of Med
for Steel Columns and Struts

l = length in inches r = least radius of gyration in inches

l/r	Rankine's (Gordon's) and Cambria	Philadelphia	Boston,* before 1919	Washington, D. C.†	Chicago ‡ and N. Y.	Am. Bridge Co. and Carnegie	Fowler's formula for struts
	$\frac{12\,500}{1+\frac{l^2}{36\,000r^2}}$	$\frac{16\,250}{1+\frac{l^2}{11\,000r^2}}$	$\frac{16\,000}{1+\frac{l^2}{20\,000r^2}}$	$15\,200-58l/r$	$16\,000-70l/r$ Chi. 14 000 max N. Y. 16 000 max	$19\,000-100l/r$ 13 000 max	$12\,500-50l/r$
I	II	III	IV	V	VI	VII	VIII
70	11 000	11 240	12 850	11 140	11 100	12 000	9 000
71	10 965	11 140	12 780	11 082	11 030	11 900	8 950
72	10 930	11 040	12 710	11 024	10 960	11 800	8 900
73	10 890	10 940	12 640	10 966	10 890	11 700	8 850
74	10 850	10 845	12 565	10 908	10 820	11 600	8 800
75	10 810	10 750	12 490	10 850	10 750	11 500	8 750
76	10 770	10 655	12 420	10 792	10 680	11 400	8 700
77	10 735	10 560	12 345	10 734	10 610	11 300	8 650
78	10 695	10 465	12 270	10 676	10 540	11 200	8 600
79	10 655	10 370	12 195	10 618	10 470	11 100	8 550
80	10 615	10 275	12 120	10 560	10 400	11 000	8 500
81	10 575	10 180	12 045	10 502	10 330	10 900	8 450
82	10 535	10 085	11 970	10 444	10 260	10 800	8 400
83	10 495	9 990	11 895	10 386	10 190	10 700	8 350
84	10 450	9 900	11 825	10 328	10 120	10 600	8 300
85	10 410	9 810	11 755	10 270	10 050	10 500	8 250
86	10 370	9 720	11 680	10 212	9 980	10 400	8 200
87	10 330	9 630	11 605	10 154	9 910	10 300	8 150
88	10 290	9 540	11 530	10 096	9 840	10 200	8 100
89	10 250	9 450	11 460	10 038	9 770	10 100	8 050
90	10 205	9 360	11 390	9 980	9 700	10 000	8 000
91	10 165	9 270	11 315	9 922	9 630	9 900	8 950
92	10 125	9 185	11 240	9 864	9 560	9 800	8 900
93	10 085	9 100	11 165	9 806	9 490	9 700	8 850
94	10 040	9 015	11 095	9 748	9 420	9 600	8 800
95	9 995	8 930	11 025	9 690	9 350	9 500	7 750
96	9 955	8 845	10 950	9 632	9 280	9 400	7 700
97	9 915	8 760	10 880	9 574	9 210	9 300	7 650
98	9 875	8 675	10 810	9 516	9 140	9 200	7 600
99	9 830	8 590	10 740	9 458	9 070	9 100	7 550
100	9 785	8 510	10 670	9 400	9 000	9 000	7 500
101	9 740	8 430	10 595	9 342	8 930	8 900	7 450
102	9 695	8 350	10 525	9 284	8 860	8 800	7 400
103	9 650	8 270	10 455	9 226	8 790	8 700	7 350
104	9 610	8 190	10 385	9 168	8 720	8 600	7 300
105	9 570	8 115	10 315	9 110	8 650	8 500	7 250
106	9 525	8 040	10 245	9 052	8 580	8 400	7 200
107	9 480	7 965	10 175	8 994	8 510	8 300	7 150
108	9 435	7 890	10 105	8 936	8 440	8 200	7 100
109	9 395	7 815	10 035	8 878	8 370	8 100	7 050

* New Boston Code, $20\,000-100\,l/r$; maximum l/r, 160; maximum stress, 1
† Also old New York Code, Newark, N. J., Atlanta, Ga., Worcester, Mass., et
‡ Am. R'y Engrg. Ass'n the same up to l/r, 100, for main members. Maximum,

Table XI (Continued). Safe Loads in Pounds per Square Inch of Metal-Area for Steel Columns and Struts

l = length in inches r = least radius of gyration in inches

l/r	Rankine's (Gordon's) and Cambria	Philadelphia	Boston,* before 1919	Washington, D. C.†	Chicago ‡ and N. Y.	Am. Bridge Co. and Carnegie	Fowler's formula for struts	l/r
	$\frac{12\,500}{1+\frac{l^2}{36\,000\,r^2}}$	$\frac{16\,250}{1+\frac{l^2}{11\,000\,r^2}}$	$\frac{16\,000}{1+\frac{l^2}{20\,000\,r^2}}$	$15\,200 - 58\ l/r$	$16\,000 - 70\ l/r$ Chi. 14 000 max N.Y. 16 000 max	$19\,000 - 100\ l/r$ 13 000 max	$12\,500 - 50\ l/r$	
I	II	III	IV	V	VI	VII	VIII	IX
110	9 355	7 740	9 970	8 820	8 300	8 000	7 000	110
111	9 320	7 665	9 900	8 762	8 230	7 900	6 950	111
112	9 265	7 590	9 830	8 704	8 160	7 800	6 900	112
113	9 220	7 520	9 760	8 646	8 090	7 700	6 850	113
114	9 180	7 450	9 695	8 588	8 020	7 600	6 800	114
115	9 140	7 380	9 630	8 530	7 950	7 500	6 750	115
116	9 095	7 310	9 560	8 472	7 880	7 400	6 700	116
117	9 050	7 240	9 495	8 414	7 810	7 300	6 650	117
118	9 010	7 170	9 430	8 356	7 740	7 200	6 600	118
119	8 970	7 100	9 365	8 298	7 670	7 100	6 550	119
120	8 930	7 035	9 300	8 240	7 600	7 000	6 500	120

In the COMPARATIVE DIAGRAM (page 496) OF COMPRESSION FORMULAS the lines of the formulas and the maximum ratio of l/r for main members and secondary members are as follows:

Compression-formulas	Maximum ratio of l/r §	
	Main members	Secondary members
American Bridge Company	120	200
American Railway Engineering Association	100	120
Chicago Building Laws	120	150
Rankine's (Gordon's) formula	200	200
New York Building Laws	120	120
Philadelphia Building Laws	140	140
Boston Building Laws (before 1919)	120	120
Boston Building Laws (since 1919)	160	160

*† ‡ See foot-notes on pages 493 and 494. § See important notes on page 490.

Comparative Diagram of Compression-Formulas

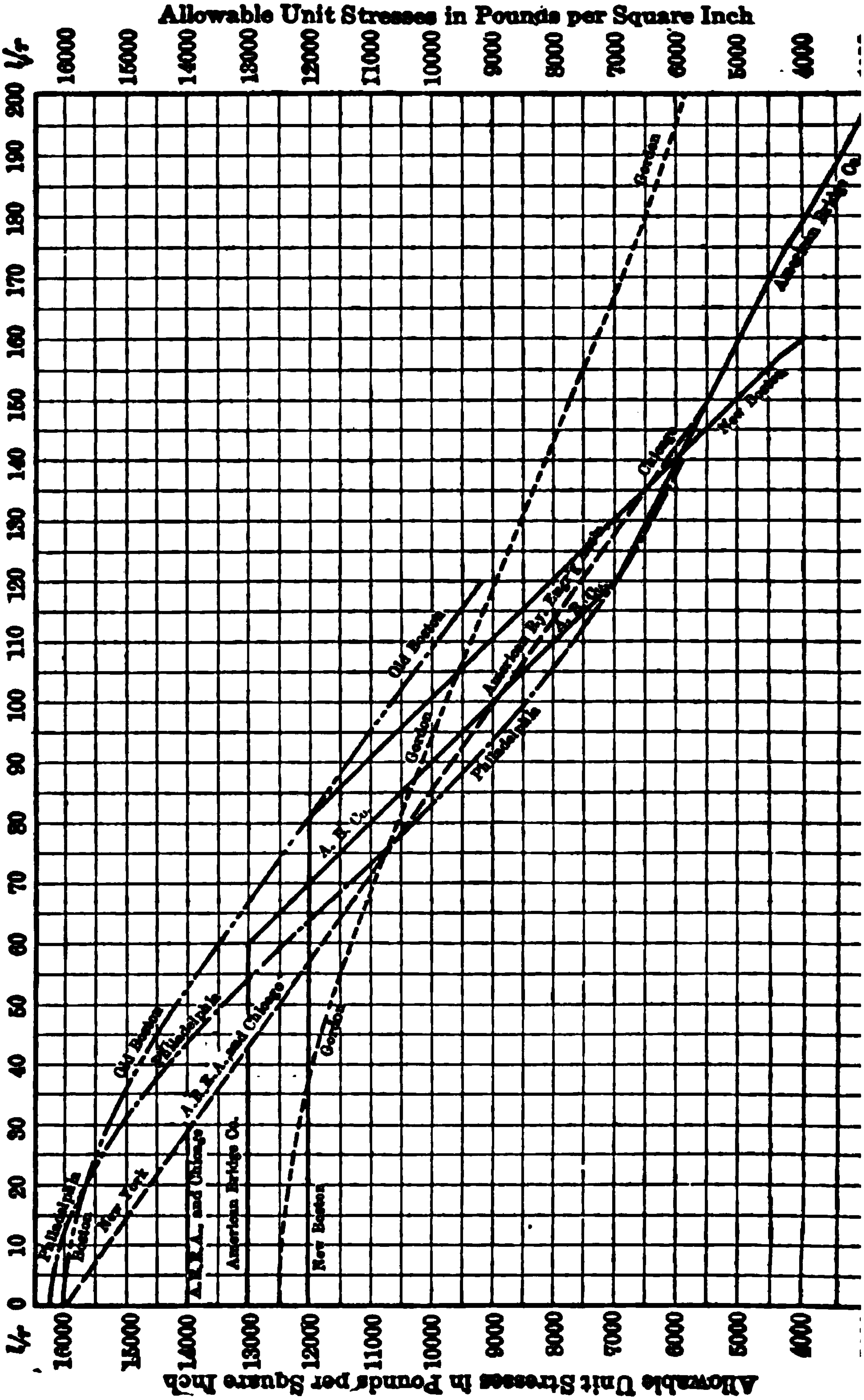


Table XII.* Safe Loads in Tons of 2 000 Pounds for Standard Steel-Pipe Columns. See National Tube Company's Handbook for Values of r Used

Loads in tons of 2 000 pounds. Table based on New York and Chicago laws. Formula used, $S = 16\,000 - 70\, l/r$, in which

S = allowable compressive stress for steel in pounds per square inch,
l = length of column in inches,
r = least radius of gyration in inches.

Loads above or to the left of the zigzag lines correspond to values of l/r greater than 120.

Length, ft	Nominal sizes of pipe. Inside diameters in inches								
	2	2½	3	3½	4	4½	5	6	7
	Thickness in decimal parts of an inch								
	0.154	0.203	0.216	0.226	0.237	0.247	0.258	0.280	0.301
40	.....	.....	.....	.....	.....	.....	.....	.....	..
36	.....	.....	.....	.....	.....	.....	.....	.....	15.00
33	.....	.....	.....	.....	.....	.....	.....	10.20	18.37
30	.....	.....	.....	.....	.....	.....	.....	13.33	21.73
27	.....	.....	.....	.....	.....	.....	8.43	16.46	25.10
24	.....	.....	.....	.....	.....	7.41	11.32	19.60	28.47
22	.....	.....	.....	.....	5.96	9.25	13.24	21.68	30.71
20	.....	.....	.....	4.60	7.73	11.09	15.16	23.77	32.96
18	.....	.....	.....	6.28	9.50	12.94	17.09	25.86	35.20
16	.....	.....	4.96	7.97	11.26	14.78	19.01	27.95	37.45
14	.....	3.06	6.57	9.65	13.03	16.62	20.93	30.04	39.69
13	.....	3.81	7.37	10.49	13.91	17.54	21.90	31.08	40.81
12	.....	4.57	8.18	11.33	14.79	18.46	22.86	32.12	41.94
11	2.29	5.32	8.98	12.18	15.68	19.38	23.82	33.17	43.06
10	2.86	6.08	9.78	13.02	16.56	20.30	24.78	34.21	44.18
9	3.44	6.83	10.59	13.86	17.44	21.22	25.74	35.26	45.30
8	4.01	7.59	11.39	14.70	18.33	22.14	26.71	36.30	46.43
7	4.58	8.34	12.20	15.54	19.21	23.06	27.67	37.34	47.55
6	5.16	9.10	13.00	16.38	20.09	23.98	28.63	38.39	48.48
5	5.73	9.86	13.81	17.23	20.98	24.90	29.59	39.07	48.48

Length, ft	Nominal sizes of pipe. Inside diameters in inches.							
	8	9	10	11	12	13	14	15
	Thickness in decimals parts of an inch							
	0.322	0.342	0.365	0.375	0.375	0.375	0.375	0.375
40	19.86	28.77	40.81	51.26	60.68	72.45	81.88	91.30
36	23.96	33.87	46.36	56.85	66.27	78.05	87.47	96.89
33	27.57	37.70	50.34	61.05	70.47	82.25	91.67	101.09
30	31.17	41.53	54.43	65.24	74.67	86.44	95.87	105.29
27	34.77	45.35	58.51	69.44	78.86	90.64	100.06	109.49
	38.37	49.18	62.59	73.64	83.06	94.84	104.26	113.68
	40.78	51.73	65.32	76.43	85.86	97.64	107.06	116.48
	43.18	54.28	68.04	79.23	88.65	100.43	109.86	119.28
	45.58	56.83	70.76	82.03	91.45	103.23	112.65	122.08
	47.98	59.38	73.48	84.83	94.25	106.03	115.45	124.88
	50.38	61.93	76.21	87.62	97.05	108.83	118.25	127.67
	51.58	63.21	77.57	89.02	98.45	110.23	119.65	128.85
	52.78	64.49	78.93	90.42	99.85	111.62	120.61	128.85
	53.99	65.76	80.29	91.82	101.24	112.36	120.61	128.85
	55.19	67.04	81.65	92.22	102.05	112.36	120.61	128.85
	56.39	68.31	83.01	93.81	102.05	112.36	120.61	128.85
	57.59	69.59	83.36	93.81	102.05	112.36	120.61	128.85
	58.79	69.82	83.36	93.81	102.05	112.36	120.61	128.85
	58.79	69.82	83.36	93.81	102.05	112.36	120.61	128.85
	58.79	69.82	83.36	93.81	102.05	112.36	120.61	128.85

* Furnished by the National Tube Company, Pittsburgh, Pa.

Table XIII.* Safe Loads in Tons of 2 000 Pounds for Extra-Strong Ste Pipe Columns. See National Tube Company's Handbook for Values of r Use

Loads in tons of 2 000 pounds. Table based on New York and Chicago laws. F mula used $S = 16\,000 - 70\,l/r$, in which
S = allowable compressive stress for steel in pounds per square inch,
l = length of column in inches,
r = least radius of gyration in inches.
Loads above or to the left of the zigzag lines correspond to values of l/r greater than 1

Lengths, ft	Nominal sizes of pipe. Inside diameters in inches								
	2	2½	3	3½	4	4½	5	6	;
	Thickness in decimal parts of an inch								
	0.218	0.276	0.300	0.318	0.337	0.355	0.375	5.432	0.;
40	.....	.....	.....	.....	.....	.....	.....	.....	..
36	.....	.....	.....	.....	.....	.....	.....	.....	22
33	.....	.....	.....	.....	.....	.....	.....	14.16	28
30	.....	.....	.....	.....	.....	.....	.....	18.99	33
27	.....	.....	.....	.....	.....	.....	11.21	23.81	39
24	.....	.....	.....	.....	.....	9.74	15.39	28.64	44
22	.....	.....	.....	.....	7.68	12.38	18.19	31.86	48
20	.....	.....	.....	5.78	10.19	15.02	20.98	35.07	52
18	.....	.....	.....	8.14	12.69	17.66	23.77	38.29	56
16	.....	.....	6.29	10.51	15.20	20.31	26.56	41.51	59
14	.....	3.69	8.52	12.87	17.71	22.95	29.35	44.72	63
13	.....	4.71	9.64	14.06	18.96	24.27	30.75	46.33	65
12	.....	5.74	10.75	15.24	20.22	25.59	32.15	47.94	67
11	2.91	6.76	11.87	16.42	21.47	26.91	33.54	49.55	69
10	3.72	7.79	12.98	17.60	22.72	28.23	34.94	51.16	70
9	4.53	8.81	14.09	18.79	23.98	29.55	36.33	52.76	72
8	5.34	9.83	15.21	19.97	25.23	30.88	37.73	54.37	74
7	6.15	10.86	16.32	21.15	26.48	32.20	39.12	55.98	76
6	6.96	11.88	17.44	22.33	27.74	33.52	40.52	57.79	78
5	7.77	12.91	18.55	23.52	28.99	34.84	41.92	58.83	78

Lengths, ft	Nominal sizes of pipe. Inside diameters in inches							
	8	9	10	11	12	13	14	1;
	Thickness in decimal parts of an inch							
	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.5
40	27.60	40.14	54.25	66.80	79.36	95.06	107.62	120
36	35.05	47.59	61.71	74.26	86.82	102.52	115.08	127
33	40.64	53.18	67.30	79.85	92.41	108.11	120.67	133
30	46.23	58.77	72.89	85.45	98.00	113.70	126.27	139
27	51.81	64.36	78.48	91.04	103.60	119.30	131.86	144
24	57.40	69.95	84.07	96.63	109.19	124.89	137.45	150
22	61.13	73.68	87.80	100.36	112.92	128.62	141.18	153
20	64.85	77.40	91.53	104.09	116.65	132.35	144.91	157
18	68.58	81.13	95.26	107.82	120.38	136.08	148.64	161
16	72.30	84.86	98.98	111.54	124.11	139.81	152.37	164
14	76.03	88.58	102.71	115.27	127.84	143.54	156.10	168
13	77.89	90.45	104.58	117.14	129.70	145.40	157.97	170
12	79.75	92.31	106.44	119.00	131.56	147.27	159.44	170
11	81.61	94.17	108.30	120.87	133.43	148.44	159.44	170
10	83.48	96.04	110.17	122.73	134.70	148.44	159.44	170
9	85.34	97.90	112.03	123.70	134.70	148.44	159.44	170
8	87.20	99.76	112.70	123.70	134.70	148.44	159.44	170
7	89.06	100.33	112.70	123.70	134.70	148.44	159.44	170
6	89.34	100.33	112.70	123.70	134.70	148.44	159.44	170
5	89.34	100.33	112.70	123.70	134.70	148.44	159.44	170

* Furnished by the National Tube Company, Pittsburgh, Pa.

Table XIV. Safe Loads* in Tons of 2 000 Pounds for Struts Formed of a Pair of Steel Channels

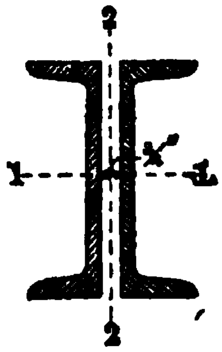
Distance between webs, ¾ in

If strut is free to bend in either direction, use smaller load given

Stresses in pounds per square inch:

12 000 for lengths of 30 radii and under;

13 500 − 50 l/r for lengths over 30 radii



Depth, in	Weight per lin foot, lb †	Thickness of web, in	Area of two channels, sq in	r_{2-2} r_{1-1}, in	Length in feet					
					8	9	10	11	12	14
15	33.9	0.40	19.80	1.48	101.57	97.56	93.55	89.54	85.48	77.44
				5.62	118.80	118.80	118.80	118.80	118.80	118.80
	35.0	0.42	20.46	1.47	105.32	101.13	96.93	92.73	88.54	80.09
				5.58	123.48	123.48	123.48	123.48	123.48	123.00
	40.0	0.52	23.40	1.46	120.13	115.30	110.48	103.66	100.78	91.14
				5.44	141.12	141.12	141.12	141.12	141.12	140.41
	45.0	0.62	26.34	1.45	134.91	129.48	123.99	118.50	113.00	102.08
				5.33	158.88	158.88	158.88	158.88	158.88	157.82
	50.0	0.72	29.28	1.46	150.36	144.23	138.20	132.17	126.06	114.00
				5.24	176.52	176.52	176.52	176.52	176.52	174.75
	55.0	0.81	32.22	1.47	165.60	159.00	152.40	145.78	139.22	126.10
				5.16	194.16	194.16	194.16	194.16	194.16	192.00
12	20.7	0.28	12.06	1.34	59.81	57.10	54.40	51.70	49.02	43.62
				4.61	72.36	72.36	72.36	72.36	71.99	70.43
	25.0	0.39	14.64	1.31	72.32	68.95	65.60	62.21	58.83	52.03
				4.43	88.20	88.20	88.20	88.20	87.28	85.26
	30.0	0.51	17.58	1.30	86.52	82.46	78.36	74.30	70.25	62.09
				4.28	105.84	105.84	105.84	105.48	104.25	101.78
	35.0	0.63	20.52	1.31	101.25	96.52	91.78	87.10	82.37	72.90
				4.18	123.48	123.48	123.48	122.65	121.16	118.33
	40.0	0.76	23.46	1.32	116.01	110.66	105.31	99.96	94.66	83.96
				4.09	141.12	141.12	141.12	139.82	138.06	134.65
10	15.3	0.24	8.94	1.24	42.94	40.78	38.64	36.48	34.32	30.01
				3.87	53.52	53.52	53.29	52.62	51.91	50.43
	20.0	0.38	11.72	1.20	55.86	52.92	49.98	47.04	44.10	38.22
				3.66	70.56	70.56	69.73	68.79	67.82	65.85
	25.0	0.53	14.66	1.20	69.82	66.15	62.47	58.80	55.12	47.77
				3.52	88.20	87.94	86.69	85.44	84.19	81.69
	30.0	0.67	17.60	1.22	84.40	80.04	75.71	71.35	67.03	58.34
				3.42	105.84	105.13	103.63	102.04	100.20	97.41
	35.0	0.82	20.54	1.26	99.76	94.82	89.93	85.04	80.16	70.33
				3.34	123.48	122.34	120.49	118.64	116.79	113.13
9	13.4	0.23	7.78	1.19	36.83	34.87	32.91	30.94	28.98	25.07
				3.49	46.68	46.48	45.82	45.16	44.50	43.15
	15.0	0.29	8.78	1.17	41.45	39.18	36.93	34.66	32.41	27.89
				3.40	52.92	52.52	51.81	50.98	50.10	48.64
	20.0	0.45	11.72	1.15	54.85	51.77	48.71	45.65	42.57	36.42
				3.22	70.56	69.50	68.38	67.29	66.20	64.00
	25.0	0.61	14.66	1.17	69.09	65.31	61.55	57.77	54.00	46.48
				3.10	87.83	86.43	85.00	83.56	82.17	79.30

* The values vary slightly with slight changes in section-areas of channels.
† Of single channel.

Table XIV (Continued). Safe Loads* in Tons of 2 000 Pounds for Stru Formed of a Pair of Steel Channels

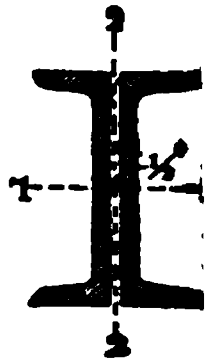
Distance between webs, ½ in

If strut is free to bend in either direction, use smaller load given

Stresses in pounds per square inch:

11 000 for lengths of 50 radii and under;

13 500 − 50 l/r for lengths over 50 radii



Depth, in	Weight per lin foot, lb †	Thickness of web, in	Area of two channels, sq in	r_{2-2} r_{1-1}, in	Length in feet 6	7	8	9	10
8	11.50	0.22	6.72	1.04	33.63	31.70	29.76	27.83	25.91
				3.10	36.85	36.85	36.85	36.85	36.85
	13.75	0.30	8.04	1.04	40.56	38.23	35.89	33.57	31.24
				2.99	44.44	44.44	44.44	44.44	44.44
	16.25	0.40	9.52	1.03	47.82	45.05	42.25	39.48	36.68
				2.89	52.58	52.58	52.58	52.58	52.58
	18.75	0.49	10.98	1.03	55.12	51.93	48.70	45.51	42.29
				2.82	60.61	60.61	60.61	60.61	60.61
	21.25	0.58	12.46	1.03	62.53	58.90	55.25	51.62	47.96
				2.77	68.75	68.75	68.75	68.75	68.75
7	9.80	0.21	5.70	0.99	28.11	26.39	24.66	22.94	21.20
				2.72	31.35	31.35	31.35	31.35	31.35
	12.25	0.31	7.16	0.99	35.51	33.33	31.15	28.98	26.78
				2.59	39.60	39.60	39.60	39.60	39.60
	14.75	0.42	8.64	0.99	42.71	40.18	37.56	34.93	32.28
				2.51	47.74	47.74	47.74	47.74	47.74
	17.25	0.52	10.10	1.00	50.19	47.15	44.10	41.06	38.02
				2.44	55.77	55.77	55.77	55.77	55.77
	19.75	0.63	11.58	1.00	57.52	54.03	50.54	47.06	43.57
				2.39	63.91	63.91	63.91	63.91	63.91
6	8.02	0.20	4.78	0.94	23.02	21.50	19.98	18.46	16.94
				2.34	26.18	26.18	26.18	26.18	26.02
	10.50	0.31	6.14	0.94	29.89	27.91	25.94	23.97	22.00
				2.22	33.99	33.99	33.99	33.99	33.32
	13.00	0.44	7.62	0.95	37.11	34.68	32.27	29.87	27.44
				2.13	42.02	42.02	42.02	41.88	40.81
	15.50	0.56	9.08	0.95	44.30	41.40	38.53	35.66	32.78
				2.07	50.16	50.16	50.16	49.68	48.33
5	6.70	0.19	3.90	0.89	18.43	17.13	15.81	14.49	13.18
				1.95	21.45	21.45	21.45	20.92	20.32
	9.00	0.33	5.26	0.90	25.17	23.41	21.65	19.87	18.11
				1.83	29.15	29.15	28.83	27.97	27.10
	11.50	0.47	6.72	0.91	32.26	30.03	27.81	25.58	23.35
				1.76	37.18	37.18	36.36	35.20	34.03
4	5.40	0.18	3.12	0.84	14.28	13.17	12.07	10.96	9.85
				1.56	17.05	16.75	16.15	15.55	14.96
	6.25	0.25	3.64	0.84	16.95	15.64	14.33	13.02	11.70
				1.50	20.24	19.72	18.99	18.26	17.53
	7.25	0.32	4.24	0.84	19.62	18.10	16.59	15.07	13.54
				1.47	23.43	22.63	21.75	20.87	19.98

* The values vary slightly with slight changes in section-areas of channels.
† Of single channel.

Table XV. Safe Loads in Tons of 2 000 Pounds for Single-Steel-Angle Struts

ANGLES WITH UNEQUAL LEGS Stresses in pounds per square inch: 11 000 for lengths of 50 radii and under; 13 500 − 50 l/r for lengths over 50 radii										
Size, in.	Thickness, in	r axis 3-3,* in	Area, sq in	Length in feet						
				4	5	6	7	8	9	10
6 X4	⅜	0.88	3.61	....	....	....	....	....	....	....
	⅞	0.86	7.99	42.78	40.00	37.21	34.44	31.64	28.86	26.07
5 X3½	⅜	0.76	3.05	....	....	....	....	....	....	....
	¾	0.75	5.81	....	....	....	....	....	....	....
5 X3	5/16	0.66	2.40	....	....	....	....	....	....	....
	¾	0.64	5.44	....	....	....	....	....	....	....
4½X3	5/16	0.66	2.25	....	....	....	....	....	....	....
	¾	0.64	5.06	24.66	22.29	19.92	17.55	15.18	....	....
4 X3½	5/16	0.73	2.25	11.49	10.57	9.65	8.72	7.79	6.86	....
	¾	0.72	5.06	25.73	23.62	21.51	19.40	17.29	15.18	....
4 X3	5/16	0.65	2.09	10.25	9.28	8.32	7.36	6.39	....	....
	¾	0.64	4.69	22.86	20.67	18.42	16.27	14.07	....	....
3½X3	5/16	0.63	1.93	9.35	8.43	7.51	6.59	....	....	....
	⅜	0.62	2.30	11.07	9.96	8.84	7.74	....	....	....
	⅝	0.62	3.67	17.67	15.90	14.12	12.35	....	....	....
3½X2½	¼	0.54	1.44	6.52	5.72	4.92	....	....	....	....
	⅜	0.54	2.11	9.55	8.38	7.21	....	....	....	....
	½	0.53	2.75	12.34	10.78	9.22	....	....	....	....
3 X2½	¼	0.53	1.31	5.88	5.13	4.39	....	....	....	....
	⅜	0.52	1.92	8.52	7.42	6.31	....	....	....	....
	½	0.52	2.50	11.10	9.66	8.22	....	....	....	....
3 X2	¼	0.43	1.19	4.71	3.88	....	....	....	....	....
	⅜	0.43	1.73	6.85	5.64	....	....	....	....	....
	½	0.43	2.25	8.91	7.34	....	....	....	....	....
2½X2	¼	0.42	1.06	4.13	3.37	....	....	....	....	....
	⅜	0.42	1.55	6.03	4.93	....	....	....	....	....
	½	0.42	2.00	7.79	6.36	....	....	....	....	....

* This is the least radius of gyration with reference to the diagonal axis 3–3. (See Table XI, pages 362 to 365.)

Table XV (Continued). Safe Loads in Tons of 2 000 Pounds for Single-Steel-Angle Struts

ANGLES WITH EQUAL LEGS

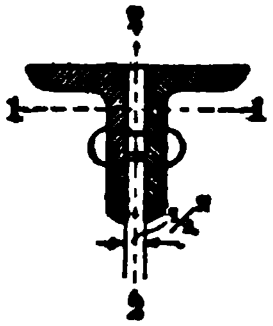
Stresses in pounds per square inch:
11 000 for lengths of 50 radii and under;
13 500 − 50 l/r for lengths over 50 radii

Size, in	Thickness, in	r axis 3–3,* in	Area, sq in	Length in feet						
				4	5	6	7	8	9	1
6 ×6	3/8	1.19	4.36	23.98	23.93	22.83	21.74	20.64	19.54	18
	5/8	1.18	7.11	39.10	38.96	37.14	35.35	33.54	31.72	29
	7/8	1.17	9.74	53.57	53.27	50.77	48.28	45.77	43.26	40
5 ×5	3/8	0.99	3.61	19.85	18.89	17.80	16.71	15.64	14.53	13
	5/8	0.97	5.86	32.23	30.50	28.68	26.86	25.06	23.24	21
	7/8	0.96	7.99	43.94	41.44	38.95	36.45	33.95	31.46	28
4 ×4	3/8	0.79	2.86	14.96	13.88	12.79	11.71	10.61	9.53	..
	1/2	0.78	3.75	19.54	18.10	16.65	15.22	13.78	12.33	..
	5/8	0.77	4.61	23.93	22.13	20.33	18.55	16.75	14.95	..
	3/4	0.77	5.44	28.24	26.12	23.99	21.89	19.77	17.65	..
3½×3½	5/16	0.69	2.09	10.47	9.56	8.65	7.74	6.83	....	..
	1/2	0.68	3.25	16.20	14.77	13.34	11.90	10.47	....	..
	5/8	0.67	3.98	19.74	17.95	16.17	14.39	12.61	....	..
	3/4	0.67	4.69	13.26	21.16	19.06	16.96	14.86	....	..
3 ×3	1/4	0.59	1.44	6.79	6.06	5.32	4.59	....	....	..
	3/8	0.58	2.11	9.88	8.78	7.69	6.60	....	....	..
	1/2	0.58	2.75	12.87	11.45	10.03	8.60	....	....	..
	5/8	0.57	3.36	15.60	13.84	12.07	10.30	....	....	..
2½×2½	5/16	0.49	0.90	3.87	3.32	2.76	....	....	....	..
	1/4	0.49	1.19	5.10	4.39	3.66	....	....	....	..
	3/8	0.48	1.73	7.35	6.27	5.19	....	....	....	..
	1/2	0.47	2.25	9.44	8.01	6.57	....	....	....	..
2¼×2¼	3/16	0.44	0.81	3.26	2.70	....	....	....	....	..
	1/4	0.44	1.06	4.26	3.54	2.80	....	....	....	..
	3/8	0.43	1.55	6.13	5.05	3.95	....	....	....	..
	7/16	0.43	1.78	7.14	5.80	4.53	....	....	....	..
2 ×2	3/16	0.40	0.72	2.70	2.16	....	....	....	....	..
	1/4	0.39	0.94	3.45	2.72	2.00	....	....	....	..

* This is the least radius of gyration, with reference to the diagonal axis 3–3. Table XII, pages 366 and 367.)

Table XVI. Safe Loads in Tons of 2 000 Pounds for Double-Steel-Angle Struts

LONG LEGS PARALLEL AND ONE-HALF INCH APART



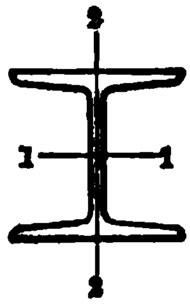
Stresses in pounds per square inch:

11 000 for lengths of 50 radii and under;

13 500 − 50 l/r for lengths over 50 radii

Size, in	Thickness, in	Least r, in	Area two angles, sq in	Length in feet						
				5	6	7	8	10	11	12
X6	½	2.49	13.52	74.36	74.36	74.36	74.36	74.36	73.34	71.72
	1	2.65	26.82	147.51	147.51	147.51	147.51	147.51	147.51	144.62
X4	⅜	1.67	7.22	39.71	39.71	39.65	38.35	35.77	34.47	33.14
	13/16	1.74	14.94	82.17	82.17	82.17	80.26	75.07	72.50	69.91
X3½	⅜	1.43	6.84	37.62	37.57	36.13	34.69	31.82	30.38	29.00
	½	1.46	9.00	49.50	49.50	47.81	45.97	42.27	40.41	38.56
	⅝	1.49	11.10	61.05	61.05	59.27	57.05	52.51	50.29	48.07
	13/16	1.52	14.12	77.66	77.66	75.82	73.03	67.42	64.65	61.88
X4	⅜	1.59	6.46	35.53	35.53	35.07	33.86	31.41	30.20	28.99
	¾	1.54	12.38	68.09	68.09	66.69	64.28	59.45	57.04	54.62
X3½	⅜	1.51	6.10	33.55	33.55	32.70	31.49	29.05	27.84	26.64
	¾	1.55	11.62	63.91	63.91	62.69	60.45	55.95	53.71	51.44
X3	⅜	1.27	5.72	31.46	30.50	29.15	27.80	25.09	23.75	22.39
	½	1.30	7.50	41.25	40.23	38.51	36.78	33.32	31.59	29.85
	⅝	1.33	9.22	50.71	49.76	47.69	45.59	41.40	39.30	37.29
	¾	1.36	10.88	59.84	58.94	56.63	54.23	49.42	47.05	44.66
X3¼	⅜	1.25	5.34	29.37	28.35	27.07	25.79	23.23	21.94	20.66
	¾	1.20	10.12	55.66	53.13	50.60	48.07	43.01	40.48	39.95
X3	⅜	1.26	4.96	27.28	26.28	25.12	24.00	21.64	20.49	19.30
	¾	1.22	9.38	51.59	49.47	47.18	44.85	40.24	37.94	35.64
X3¼	¼	1.12	2.88	15.58	14.81	14.03	13.26	11.72	10.95	10.18
	⅜	1.10	4.22	22.73	21.59	20.42	19.28	16.97	15.82	14.67
	½	1.09	5.50	29.56	28.05	26.53	25.02	21.98	20.47	18.96
	11/16	1.06	7.30	38.92	36.86	34.78	32.74	28.58	26.51	24.43
X2	¼	0.93	2.38	12.22	11.45	10.69	9.92	8.39	7.62	....
	½	0.92	4.50	23.04	21.58	20.10	18.64	15.70	14.23	....
X2	3/16	0.79	1.62	7.86	7.24	6.63	6.01	....	....	....
	½	0.75	4.00	19.00	17.40	15.80	14.20	....	....	....
X2	3/16	0.62	1.44	6.22	5.54	4.82	4.13	....	....	....
X2	¼	0.61	1.88	8.08	7.14	6.20	5.26	....	....	....

Table XVII.* Safe Loads in Units of 1 000 Pounds for Steel-Beam Columns



Allowable fiber-stress in pounds per square inch
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, b
Formula (13),

$$S = 19\,000 - 100l/r$$

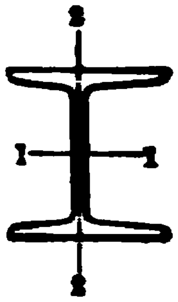
Weights do not include details
For values for l/r above 120, see notes on page 490

Effective length, ft	H beams 8-in 34.3-lb	H beams 6-in 24.1-lb	H beams 5-in 18.9-lb	H beams 4-in 13.8-lb	I beams 15-in 42.9-lb	I beams 12-in 31.8-lb	I beams 1(25
2	130.0	91.0	71.5	52.0	162.2	120.4	
3	130.0	91.0	71.5	52.0	162.2	120.4	
4	130.0	91.0	71.5	52.0	162.2	120.4	
5	130.0	91.0	71.5	50.7	162.2	120.4	
6	130.0	91.0	71.5	45.7	153.9	109.9	
7	130.0	91.0	66.0	40.6	140.1	98.9	
8	130.0	86.7	60.5	35.6	126.2	87.9	
9	130.0	80.9	55.0	30.5	112.3	76.9	
10	125.8	75.1	49.5	26.7	98.5	65.9	
11	119.4	69.3	44.0	24.2	86.0	59.9	
12	113.0	63.5	38.5	21.7	79.0	54.4	
13	106.6	57.7	35.8	19.2	72.1	48.9	
14	100.2	51.9	33.0	16.6	65.2	43.4	
15	93.8	47.6	30.3	14.1	58.2	37.9	
16	87.3	44.7	27.5	....	51.3	32.4	
17	80.9	41.8	24.8	....	44.4	26.9	..
18	74.5	38.9	22.0	....	37.4	....	..
19	69.0	36.0	19.3	....	....	....	..
20	65.8	33.1	16.5	....	....	....	..
21	62.6	30.2	....	....	....	....	.
22	59.4	27.3	....	....	....	....	.
23	56.2	24.4	....	....	....	....	.
24	53.0	21.5	....	....	....	....	.
25	49.8	....	....	....	....	....	.
26	46.6	....	....	....	....	....	.
27	43.4	....	....	....	....	....	.
28	40.2	....	....	....	....	....	.
29	37.0	....	....	....	....	....	.
30	33.7	....	....	....	....	....	.
31	30.5	....	....	....	....	....	.
Area, sq in	10.01	7.01	5.47	4.00	12.49	9.26	
I_{1-1}, in^4....	115.4	45.1	23.8	10.7	441.8	215.8	
r_{1-1}, in....	3.40	2.54	2.08	1.63	5.95	4.83	
I_{2-2}, in^4....	35.1	14.7	7.9	3.6	14.6	9.5	
r_{2-2}, in....	1.87	1.45	1.20	0.95	1.08	1.01	
Weight, lb per lin ft	34.3	24.1	18.9	13.8	42.9	31.8	

Safe load-values above the upper heavy line are for ratios of l/r not ov those between the heavy lines for ratios up to 120 l/r; and those belo lower heavy line are for ratios not over 200 l/r.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XVII * (Continued). Safe Loads in Units of 1 000 Pounds for Steel-Beam Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100l/r$$

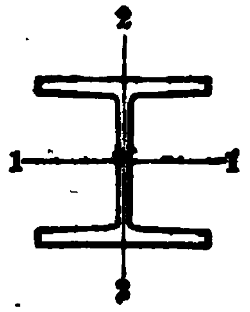
Weights do not include details
For values for l/r above 120, see notes on page 490

Effective length, ft	Depth and weight of sections					
	I beams					
	9-in 21.8-lb	8-in 18.4-lb	7-in 15.3-lb	6-in 12.5-lb	5-in 10-lb	4-in 7.7-lb
2	82.0	69.3	57.5	46.9	37.3	28.7
3	82.0	69.3	57.5	46.9	37.3	28.5
4	82.0	69.3	56.8	44.5	33.3	24.0
5	77.8	63.2	50.0	38.5	28.0	19.5
6	69.4	55.6	43.2	32.5	23.7	15.2
7	61.0	48.0	36.4	26.5	18.8	13.0
8	52.6	40.4	30.3	22.9	16.1	10.8
9	44.2	35.0	26.9	19.9	13.5	8.5
10	40.0	31.2	23.5	16.8	10.8	....
11	35.8	27.4	20.1	13.8	....	....
12	31.5	23.6	16.7	10.8	....	....
13	27.3	19.8	13.3	....	....	....
14	23.1	16.0	....	....	....	....
15	18.9	....	....	....	....	....
16	....	....	....	....	....	....
17	....	....	....	....	....	....
18	....	....	....	....	....	....
19	....	....	....	....	....	....
20	....	....	....	....	....	....
21	....	....	....	....	....	....
22	....	....	....	....	....	....
23	....	....	....	....	....	....
24	....	....	....	....	....	....
25	....	....	....	....	....	....
26	....	....	....	....	....	....
27	....	....	....	....	....	....
28	....	....	....	....	....	....
29	....	....	....	....	....	....
30	....	....	....	....	....	....
31	....	....	....	....	....	....
Area, sq in	6.32	5.34	4.43	3.61	2.87	2.21
I_1, in^4....	84.9	56.9	36.2	21.8	12.1	6.00
r_1, in.....	3.67	3.27	2.86	2.46	2.05	1.64
I_2, in^4....	5.2	3.8	2.7	1.9	1.2	0.77
r_2, in.....	0.90	0.84	0.78	0.72	0.65	0.59
Weight, per lin ft	21.8	18.4	15.3	12.5	10	7.7

Safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines for ratios up to 120 l/r; and those below the lower heavy line of ratios not over 200 l/r.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XVIII.* Safe Loads in Tons of 2 000 Pounds for Bethlehem Rol Steel 8-Inch H Columns with Square Ends



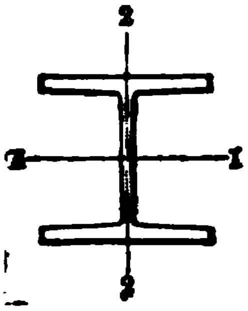
Allowable stress in pounds per square inc
13 000 for lengths under 55 radii;
16 000 − 55 l/r for lengths over 55 radii

Unsupported length, ft						
8	59.7	66.1	74.8	83.4	92.2	101.0
9	59.7	66.1	74.8	83.4	92.2	101.0
10	58.1	64.7	73.3	81.9	90.6	99.5
11	56.5	63.0	71.4	79.8	88.3	97.0
12	55.0	61.3	69.6	77.7	86.1	94.5
13	53.5	59.7	67.7	75.7	83.8	92.1
14	52.0	58.0	65.8	73.6	81.5	89.6
15	50.4	56.3	64.0	71.5	79.2	87.1
16	48.9	54.6	62.1	69.4	76.9	84.6
17	47.4	53.0	60.2	67.4	74.6	82.2
18	45.9	51.3	58.4	65.3	72.4	79.7
20	42.8	48.0	54.6	61.1	67.8	74.7
22	39.7	44.6	50.9	57.0	63.2	69.8
24	36.7	41.3	47.1	52.8	58.7	64.8
26	....	38.0	43.4	48.7	54.1	59.9
Area, sq in	9.17	10.17	11.50	12.83	14.18	15.53
I_{1-1}, in⁴.....	105.7	121.5	139.5	158.3	177.7	197.8
r_{1-1}, in.....	3.40	3.46	3.48	3.51	3.54	3.57
I_{2-2}, in⁴.....	35.8	41.1	47.2	53.4	59.8	66.3
r_{2-2}, in.....	1.98	2.01	2.03	2.04	2.05	2.07
Weight of section, lb per lin ft	32.0	34.5	39.0	43.5	48.0	53.0

Loads below the heavy line are for lengths greater than 125 radii

* See Supplementary Sections in Pamphlet S-10, published by the Bethlehem Company in March, 1921.

Table XVIII * (Continued). Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled-Steel 8-Inch H Columns with Square Ends



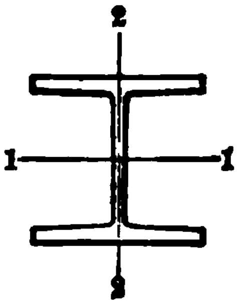
Allowable stress in pounds per square inch:
13 000 for lengths under 55 radii;
16 000 − 55 l/r for lengths over 55 radii

Unsupported length, ft							
8	118.8	127.8	136.8	146.0	154.6	163.8	173.2
9	118.8	127.8	136.8	146.0	154.6	163.8	173.2
10	117.3	126.5	135.6	144.9	153.6	163.1	172.6
11	114.4	123.5	132.4	141.4	149.9	159.3	168.6
12	111.5	120.4	129.1	137.9	146.2	155.4	164.5
13	108.7	117.3	125.8	134.4	142.6	151.6	160.5
14	105.8	114.2	122.5	131.0	138.9	147.7	156.4
15	102.9	111.2	119.2	127.5	135.2	143.9	152.4
16	100.0	108.1	116.0	124.0	131.6	140.0	148.3
17	97.1	105.0	112.7	120.5	127.9	136.1	144.3
18	94.2	101.9	109.4	117.0	124.2	132.3	140.2
20	88.5	95.8	102.9	110.1	116.9	124.6	132.1
22	82.7	89.6	96.3	103.1	109.6	116.9	124.0
24	76.9	83.5	89.8	96.2	102.2	109.2	115.9
26	71.2	77.3	83.2	89.2	94.9	101.5	107.8
Area, sq in	18.27	19.66	21.05	22.46	23.78	25.20	26.64
I_{1-1}, in^4.....	240.2	262.5	285.6	309.5	333.5	359.0	385.3
r_{1-1}, in.....	3.63	3.65	3.68	3.71	3.75	3.77	3.80
I_{2-2}, in^4.....	80.0	87.1	94.4	101.9	109.2	117.2	125.1
r_{2-2}, in.....	2.09	2.11	2.12	2.13	2.14	2.16	2.17
Weight of section, lb per lin ft	62.0	67.0	71.5	76.5	81.0	85.5	90.5

Loads below the heavy line are for lengths greater than 125 radii

* See Supplementary Sections in Pamphlet S-10, published by the Bethlehem Steel Company in March, 1921.

Table XIX.* Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled Steel 10-Inch H Columns with Square Ends



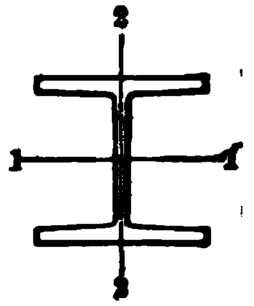
Allowable stress in pounds per square inch:
13000 for lengths under 55 radii;
16 000 − 55 l/r for lengths over 55 radii

Unsupported length, ft							
10	93.5	103.4	114.2	125.0	135.9	146.8	157
11	93.5	103.4	114.2	125.0	135.9	146.8	157
12	92.1	102.2	113.1	123.9	134.9	145.9	157
13	90.2	100.1	110.8	121.4	132.2	143.0	153
14	88.3	98.0	108.5	118.9	129.5	140.1	150
15	86.3	95.9	106.2	116.4	126.9	137.2	147
16	84.5	93.8	103.9	113.9	124.2	134.3	144
18	80.7	89.6	99.3	108.9	118.8	128.5	138
20	76.9	85.4	94.7	103.9	113.4	122.7	132
22	73.1	81.3	90.1	98.9	108.0	116.9	126
24	69.3	77.1	85.6	93.9	102.6	111.1	119
26	65.4	72.9	81.0	88.9	97.2	105.3	113
28	61.6	68.7	76.4	83.9	91.8	99.5	107
30	57.8	64.5	71.8	78.9	86.4	93.7	101
32	54.0	60.3	67.2	73.9	80.1	87.9	94
Area, sq in	14.37	15.91	17.57	19.23	20.91	22.59	24.
I_{1-1}, in^4.....	263.5	296.8	331.9	368.0	405.2	443.6	483
r_{1-1}, in.....	4.28	4.32	4.35	4.37	4.40	4.43	4.
I_{2-2}, in^4.....	89.1	100.4	112.2	124.2	136.5	149.1	16
r_{2-2}, in.....	2.49	2.51	2.53	2.54	2.56	2.57	2.
Weight of section, lb per lin ft	49.0	54.0	59.5	65.5	71.0	77.0	8

Loads below the heavy line are for lengths greater than 125 radii

* See Supplementary Sections in Pamphlet S-10, published by the Bethlehem Company in March, 1921.

Table XIX * (Continued). Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled-Steel 10-Inch H Columns with Square Ends



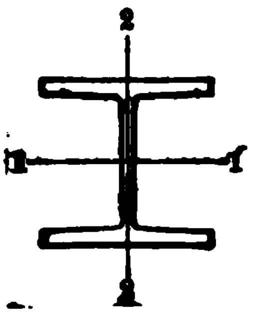
Allowable stress in pounds per square inch:
13 000 for lengths under 55 radii;
16 000 − 55 l/r for lengths over 55 radii

Unsupported length, ft							
10	168.9	180.1	190.6	201.9	213.2	224.6	236.1
11	168.9	180.1	190.6	201.9	213.2	224.6	236.1
12	168.3	179.6	190.2	201.9	213.2	224.6	236.1
13	165.0	176.1	186.6	198.0	209.3	220.7	232.2
14	161.7	172.6	182.9	194.1	205.2	216.4	227.7
15	158.4	169.1	179.2	190.3	201.2	212.1	223.2
16	155.1	165.6	175.5	186.4	197.0	207.8	218.7
18	148.5	158.6	168.1	178.6	188.9	199.2	209.8
20	142.0	151.6	160.7	170.8	180.7	190.7	200.8
22	135.4	144.6	153.3	163.1	172.5	182.1	191.8
24	128.8	137.6	145.9	155.3	164.4	173.5	182.8
26	122.2	130.6	138.5	147.5	156.2	165.0	173.8
28	115.6	123.6	131.2	139.8	148.0	156.4	164.9
30	109.0	116.6	123.8	132.0	139.9	147.8	155.9
32	102.4	109.6	116.4	124.2	131.7	139.2	146.9
Area, sq in	25.99	27.71	29.32	31.06	32.80	34.55	36.32
I_1, in^4	523.5	565.2	607.0	651.0	696.2	742.7	790.4
r_1, in	4.49	4.52	4.55	4.58	4.61	4.64	4.67
I_2, in^4	175.1	188.6	201.7	215.6	229.9	244.4	259.3
r_2, in	2.60	2.61	2.62	2.64	2.65	2.66	2.67
Weight of section, lb per lin ft	88.5	94.0	99.5	105.5	111.5	117.5	123.5

Loads below the heavy line are for lengths greater than 125 radii

* See Supplementary Sections in Pamphlet S-10, published by the Bethlehem Steel Company in March, 1921.

Table XX.* Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled-Steel 12-Inch H Columns with Square Ends



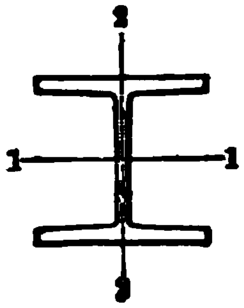
Allowable stress in pounds per square inch:
13 000 for lengths under 55 radii;
16 000 − 55 l/r for lengths over 55 radii

Unsupported length, l ft								
10	123.5	136.2	149.1	162.0	175.0	188.0	201.1	21.
12	123.5	136.2	149.1	162.0	175.0	188.0	201.1	21.
14	122.5	135.4	148.3	161.4	174.5	187.7	201.0	21.
16	118.3	130.8	143.3	155.9	168.6	181.5	194.3	20
18	114.1	126.2	138.3	150.5	162.8	175.2	187.7	20
20	109.9	121.6	133.2	145.1	156.9	169.0	181.0	19
22	105.7	117.0	128.2	139.7	151.1	162.8	174.4	18
24	101.5	112.4	123.2	134.2	145.2	156.5	167.7	17
26	97.3	107.8	118.1	128.8	139.4	150.3	161.1	17
28	93.1	103.1	113.1	123.4	133.5	144.0	154.4	16
30	88.9	98.5	108.1	117.9	127.7	137.8	147.8	15
32	84.7	93.9	103.0	112.5	121.9	131.6	141.1	15
34	80.5	89.3	98.0	107.1	116.0	125.3	134.4	14
36	76.3	84.7	93.0	101.7	110.2	119.1	127.8	13
38	......	......	......	......	104.3	112.8	121.1	12
Area, sq in	19.00	20.96	22.94	24.92	26.92	28.92	30.94	3
I_{1-1}, in^4.....	499.0	556.6	615.6	676.1	738.1	801.7	866.8	9
r_{1-1}, in.....	5.13	5.15	5.18	5.21	5.24	5.27	5.30	
I_{2-2}, in^4.....	168.6	188.2	208.1	228.5	249.2	270.1	291.7	3
r_{2-2}, in.....	2.98	3.00	3.01	3.03	3.04	3.06	3.07	
Weight of section, lb per lin ft	64.5	71.5	78.0	84.5	91.5	98.5	105.0	1

Loads below the heavy line are for lengths greater than 125 radii

* See Supplementary Sections in Pamphlet S-10, published by the Bethlehem Company in March, 1921.

Table XI* (Continued). Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled-Steel 12-Inch H Columns with Square Ends



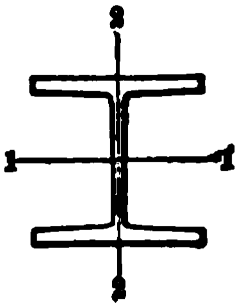
Allowable stress in pounds per square inch:
13 000 for lengths under 55 radii;
16 000 − 55 l/r for lengths over 55 radii

Unsupported length, ft							
10	226.7	239.9	253.3	266.7	280.2	293.7	307.3
12	226.7	239.9	253.3	266.7	280.2	293.7	307.3
14	226.7	239.9	253.3	266.7	280.2	293.7	307.3
16	219.6	232.6	246.0	259.3	272.6	286.0	299.7
18	212.1	224.8	237.8	250.6	263.5	276.6	289.9
20	204.7	217.0	229.6	242.0	254.5	267.1	280.1
22	197.3	209.1	221.4	233.4	245.5	257.7	270.3
24	189.9	201.3	213.2	224.8	236.4	248.3	260.5
26	182.5	193.5	204.9	216.1	227.4	238.8	250.7
28	175.0	185.6	196.7	207.5	218.4	229.4	240.9
30	167.6	177.8	188.5	198.9	209.3	219.9	231.0
32	160.2	170.0	180.3	190.3	200.3	210.5	221.2
34	152.8	162.1	172.1	181.6	191.3	201.1	211.4
36	145.3	154.3	163.9	173.0	182.3	191.6	201.6
38	137.9	146.5	155.6	164.4	173.2	182.2	191.8
Area, sq in	34.87	36.91	38.97	41.03	43.10	45.19	47.28
I_{1-1}, in⁴	1 000.0	1 069.8	1 141.3	1 214.5	1 289.4	1 366.0	1 444.3
r_{1-1}, in	5.36	5.38	5.41	5.44	5.47	5.50	5.63
I_{2-2}, in⁴	335.0	357.7	380.7	404.1	428.0	452.2	477.0
r_{2-2}, in	3.10	3.11	3.13	3.14	3.15	3.16	3.18
Weight of section, lb per lin ft	118.5	125.5	132.5	139.5	146.5	153.5	161.0

Loads below the heavy line are for lengths greater than 125 radii

*See Supplementary Sections in Pamphlet S-10, published by the Bethlehem Steel Company in March, 1921.

Table XXI.* Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled Steel 14-Inch H Columns with Square Ends



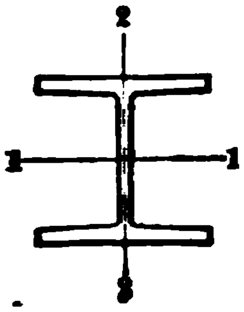
Allowable stress in pounds per square inc
13 000 for lengths under 55 radii;
16 000 − 55 l/r for lengths over 55 radii

Unsupported length, ft							
10	159.0	173.9	188.9	204.0	219.1	234.3	24
12	159.0	173.9	188.9	204.0	219.1	234.3	24
14	159.0	173.9	188 9	204.0	219.1	234.3	24
16	158.5	173.9	188.6	204.0	219.1	234.3	24
18	153.8	168.5	183.2	198.1	212.9	228.0	24
20	149.2	163.4	177.7	192.2	206.6	221.3	23
22	144.5	158.4	172.2	186.3	200.3	214.6	23
24	139.9	153.3	166.7	180.4	194.0	207.9	22
26	135.2	148.2	161.2	174.6	187.7	201.2	21
28	130.5	143.2	155.8	168.7	181.4	194.5	20
30	125.9	138.1	150.3	162.8	175.1	187.8	20
32	121.2	133.1	144.8	156.9	168.8	181.1	19
36	111.9	122.9	133.8	145.1	156.2	167.7	17
40	102.6	112.8	122.9	133.4	143.6	154.3	16
44	....	....	111.9	121.6	131.0	140.9	15
Area, sq in	24.46	26.76	29.06	31.38	33.70	36.04	3
I_{1-1}, in^4.....	884.9	976.8	1 070.6	1 166.6	1 264.5	1 364.6	1 4
r_{1-1}, in......	6.01	6.04	6.07	6.10	6.13	6.16	6
I_{2-2}, in^4.....	294.5	325.4	356.9	387.8	420.3	453.4	4
r_{2-2}, in.....	3.47	3.49	3.50	3.52	3.53	3.55	3
Weight of section, lb per lin ft	83.5	91.0	99.0	106.5	114.5	122.5	1

Loads below the heavy line are for lengths greater than 125 radii

* See Supplementary Sections in Pamphlet S-10, published by the Bethlehem Company in March, 1921.

Table XXI * (Continued). Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled-Steel 14-Inch H Columns with Square Ends



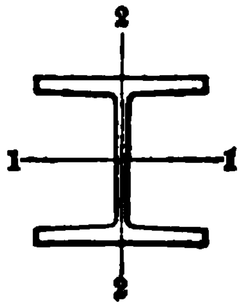
Allowable stress in pounds per square inch:
13 000 for lengths under 55 radii;
16 000 − 55 l/r for lengths over 55 radii

Unsupported length, ft						
10	263.8	279.2	294.7	310.1	325.7	341.3
12	263.8	279.2	294.7	310.1	325.7	341.3
14	263.8	279.2	294.7	310.1	325.7	341.3
16	263.8	279.2	294.7	310.1	325.7	341.3
18	257.4	272.5	288.1	303.4	319.0	334.6
20	249.9	264.6	279.8	294.7	309.9	325.1
22	242.4	256.7	271.5	286.0	300.8	315.6
24	234.9	248.9	263.2	277.3	291.7	306.1
26	227.4	241.0	254.9	268.6	282.6	296.6
28	220.0	233.1	246.6	259.9	273.5	287.1
30	212.5	225.2	238.3	251.2	264.4	277.6
32	205.0	217.3	230.0	242.5	255.3	268.1
36	190.0	201.5	213.5	225.1	237.1	249.1
40	175.1	185.7	196.9	207.7	218.9	230.1
44	160.1	170.0	180.3	190.3	200.7	211.1
Area, sq in	40.59	42.95	45.33	47.71	50.11	52.51
I_{1-1}, in^4	1 568.4	1 674.7	1 783.3	1 894.0	2 007.0	2 122.3
r_{1-1}, in	6.21	6.24	6.27	6.30	6.33	6.36
I_{2-2}, in^4	519.7	554.4	589.5	626.1	662.3	699.0
r_{2-2}, in	3.58	3.59	3.61	3.62	3.64	3.65
Weight of section, per lin ft	138.0	146.0	154.0	162.0	170.5	178.5

Loads below the heavy line are for lengths greater than 125 radii

* See Supplementary Sections in Pamphlet S-10, published by the Bethlehem Steel Company in March, 1921.

Table XXI * (Continued). Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled-Steel 14-Inch H Columns with Square Ends



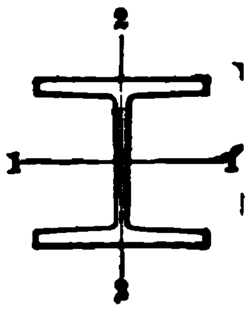
Allowable stress in pounds per square inch
13 000 for lengths under 55 radii;
16 000 − 55 l/r for lengths over 55 radii

Unsupported length, ft							
10	357.0	372.8	388.6	403.5	419.4	435.4	45
12	357.0	372.8	388.6	403.5	419.4	435.4	45
14	357.0	372.8	388.6	403.5	419.4	435.4	45
16	357.0	372.8	388.6	403.5	419.4	435.4	45
18	350.3	366.1	381.9	396.9	412.9	429.0	44
20	340.4	355.8	371.2	385.8	410.5	417.2	43
22	330.5	345.5	360.5	374.8	390.0	405.3	42
24	320.6	335.2	349.8	363.7	378.5	393.4	40
26	310.7	324.9	339.1	352.6	367.1	381.6	39
28	300.8	314.6	328.4	341.6	355.6	369.7	38
30	290.9	304.3	317.7	330.5	344.1	357.8	37
32	281.0	294.0	307.0	319.4	332.6	345.9	35
36	261.2	273.4	285.6	297.3	309.7	322.2	33
40	241.4	252.8	264.2	275.1	286.8	298.5	31
44	221.6	232.2	242.8	253.0	263.8	274.8	28
Area, sq in	54.92	57.35	59.78	62.07	64.52	66.98	69
I_{1-1}, in^4.....	2 239.8	2 359.7	2 481.9	2 603.3	2 730.2	2 859.6	2 99
r_{1-1}, in.....	6.39	6.41	6.44	6.48	6.51	6.53	[illegible]
I_{2-2}, in^4.....	736.3	744.2	812.6	849.8	889.3	929.4	97
r_{2-2}, in.....	3.66	3.67	3.69	3.70	3.71	3.73	3
Weight of section, lb per lin ft	186.5	195.0	203.5	211.0	219.5	227.5	23

Loads below the heavy line are for lengths greater than 125 radii

* See Supplementary Sections in Pamphlet S-10, published by the Bethlehem Company in March, 1921.

Table XXI* (Continued). Safe Loads in Tons of 2 000 Pounds for Bethlehem Rolled-Steel 14-Inch H Columns with Square Ends



Allowable stress in pounds per square inch:
13 000 for lengths under 55 radii;
16 000 − 55 l/r for lengths over 55 radii

Unsupported length, ft						
10	467.6	483.8	500.0	516.4	532.8	549.3
12	467.6	483.8	500.0	516.4	532.8	549.3
14	467.6	483.8	500.0	516.4	532.8	549.3
16	467.6	483.8	500.0	516.4	532.8	549.3
18	461.5	477.9	494.4	510.9	527.6	544.3
20	448.9	464.8	480.9	497.0	513.3	529.6
22	436.2	451.8	467.4	483.2	499.1	515.0
24	423.6	438.7	454.0	469.3	484.8	500.4
26	410.9	425.7	440.5	455.5	470.6	485.7
28	398.2	412.6	427.1	441.6	456.3	471.1
30	385.6	399.6	413.6	427.8	442.1	456.4
32	372.9	386.5	400.2	413.9	427.8	441.8
36	347.6	360.4	373.3	386.3	399.4	412.5
40	322.2	334.3	346.4	358.6	370.9	383.3
44	296.9	308.1	319.5	330.9	342.4	354.0
Area, sq in	71.94	74.43	76.93	79.44	81.97	84.50
I_{1-1}, in^4	3 125.8	3 262.7	3 402.1	3 544.1	3 688.8	3 836.1
r_{1-1}, in	6.59	6.62	6.65	6.68	6.71	6.74
I_{2-2}, in^4	1 011.3	1 053.2	1 095.6	1 138.7	1 182.4	1 226.7
r_{2-2}, in	3.75	3.76	3.77	3.79	3.80	3.81
Weight of section, lb per lin ft	244.5	253.0	261.5	270.0	278.5	287.5

Loads below the heavy line are for lengths greater than 125 radii

* See Supplementary Sections in Pamphlet S-10, published by the Bethlehem Steel Company in March, 1921.

Table XXII. Safe Loads in Tons of 2 000 Pounds for Light-Weight L Columns *

Factor of safety between 4.5 and 5

Calculated by the formula, $P = A_s(13\ 500 - 140\ l/d) + A_c(1\ 000 - 11\ l/d$

in which A_s and A_c are the areas of steel pipe and concrete, in square inches, length in inches, and d the outside diameter of pipe, in inches

Outside diameter, in	Weight per linear foot, in	Length of column in feet									
		6	7	8	9	10	11	12	13	14	15
3	9.64	6	6	5	....	...	...	...	...	...	...
3½	13.09	9	9	8	8	7	...	...	...	...	...
4	17.02	13	13	12	12	11	10	...	...	...	...
4½	21.05	14	14	13	13	12	11	10	...	...	...
5	25.90	20	20	19	19	18	18	17	17	16	...
6	36.82	28	28	27	27	26	26	25	24	23	23

Table XXIII. Safe Loads in Tons of 2 000 Pounds for Heavy-We Lally Columns*

Factor of safety between 4.5 and 5

These loads can be greatly increased by reinforcing the concrete

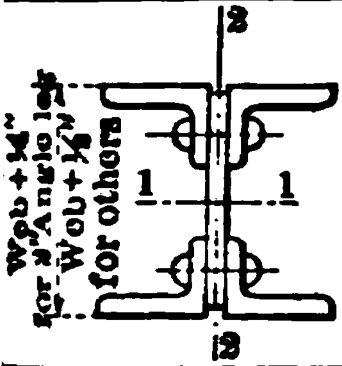
Calculated by the formula, $P = A_s(13\ 500 - 140\ l/d) + A_c(1\ 000 - 11\ l/$

in which A_s and A_c are the areas of steel pipe and concrete, in square inches, length in inches, and d the outside diameter of pipe, in inches

Outside diameter, in	Weight per linear foot, lb	Length of columns in feet						
		6	8	10	12	14	16	18
3½	15	12	11	10	9	.....	.....	.....
4	20	16	15	14	12	11	.....	.....
4½	24	20	18	17	16	15	.....	.....
5	29	27	26	24	22	21	19	.....
5½	36	32	31	29	28	26	24	22
6⅝	49	45	43	41	40	38	35	34
7⅝	64	58	56	54	52	51	49	46
8⅝	81	74	72	69	67	65	62	60
9⅝	100	93	89	87	85	82	79	77
10¾	123	111	109	107	104	101	99	96
11¾	146	131	128	124	122	119	117	113
12¾	169	150	146	144	141	139	135	133

*For areas of cross-sections of metal and concrete, and for other data used tations for determining safe loads by formula, see Handbook of the United Stat Company, Cambridge, Mass. (See, also, pages 469 to 474, and page 477.)

Table XXIV.* Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

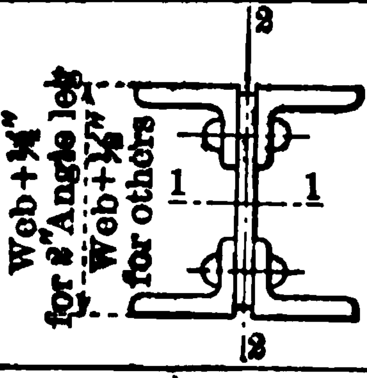
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 6"×¼"			Web-plate 8"×¼"			
	4 angles 2½"×2"×¼"	4 angles 3"×2"×¼"	4 angles 3"×2½"×¼"	4 angles 3"×2½"×¼"	4 angles 3"×2½"×5/16"	4 angles 3½"×2½"×¼"	4 angles 3½"×2½"×5/16"
6	69	81	88	94	110	101	119
7	63	78	82	86	103	101	119
8	56	72	76	79	95	96	115
9	49	66	69	72	87	89	107
10	43	60	63	65	78	83	100
11	38	54	56	57	70	76	92
12	35	49	50	50	62	70	85
13	32	43	45	47	56	63	78
14	28	40	42	43	52	57	70
15	25	37	39	39	48	52	63
16	22	34	35	36	44	49	60
17	18	32	32	32	40	46	56
18	....	29	29	28	36	43	52
19	....	26	26	25	32	39	49
20	....	23	22	....	28	36	45
21	....	20	....	....	....	33	41
22	....	....	....	....	....	30	38
23	....	....	....	....	....	27	34
24	....	....	....	....	....	23	30
25	....	....	....	....	....	....	....
26	....	....	....	....	....	....	....
27	....	....	....	....	....	....	....
28	....	....	....	....	....	....	....
29	....	....	....	....	....	....	....
30	....	....	....	....	....	....	....
…, sq in	5.74	6.26	6.74	7.24	8.48	7.76	9.12
…, in⁴.....	34.3	39.1	42.6	81.2	96.9	90.1	107
…, in.....	2.45	2.50	2.51	3.35	3.38	3.41	3.43
…, in⁴.....	6.2	10.3	10.3	10.3	12.9	16.0	20.2
…, in.....	1.04	1.28	1.24	1.19	1.23	1.44	1.49
…ight, …lin ft..	19.6	21.5	23.1	24.8	29.2	26.4	31.2

…e safe load-values above the upper heavy line are for ratios of l/r not over 60; …e between the heavy lines are for ratios up to 120 l/r; and those below the lower …y line are for ratios not over 200 l/r

*From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plat and-Angle Columns



Allowable fiber-stress in pounds per square inc
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, Formula (13),

$$S = 19\,000 - 100\,l/r$$

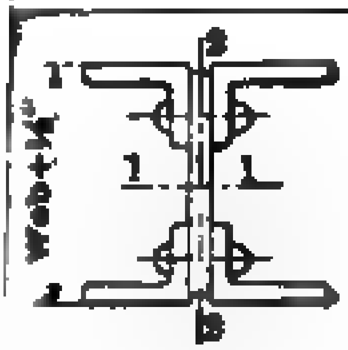
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 8"×5/16" 4 angles 3½"×2½"×5/16"	Web-plate 8"×5/16" 4 angles 3½"×2½"×⅜"	Web-plate 8"×5/16" 4 angles 4"×3"×5/16"	Web-plate 8"×5/16" 4 angles 4"×3"×⅜"	Web-plate 8"×⅜" 4 angles 4"×3"×⅜"	Web-plate 8"×⅜" 4 angles 4"×3"×7/16"	Web-plate 8"×⅜" 4 angles [illegible]
6	125	142	141	161	168	188	20[illegible]
7	125	142	141	161	168	188	20[illegible]
8	120	138	141	161	168	188	20[illegible]
9	112	130	136	158	163	185	20[illegible]
10	104	121	128	149	154	175	19[illegible]
11	96	112	121	140	145	165	18[illegible]
12	89	104	113	131	136	155	17[illegible]
13	81	95	105	123	127	145	16[illegible]
14	73	86	97	114	118	135	15[illegible]
15	66	77	89	105	109	124	14[illegible]
16	62	73	81	97	100	114	13[illegible]
17	58	68	75	88	90	104	12[illegible]
18	54	64	71	83	86	98	11[illegible]
19	50	60	67	79	81	93	10[illegible]
20	47	55	63	74	77	88	10[illegible]
21	43	51	59	70	72	83	[illegible]
22	39	47	55	66	68	78	[illegible]
23	35	42	51	61	63	73	[illegible]
24	31	38	48	57	59	68	[illegible]
25	....	34	44	53	54	63	[illegible]
26	....	....	40	48	49	58	[illegible]
27	....	....	36	44	45	53	[illegible]
28	....	....	....	39	40	48	[illegible]
29	....	....	....	....	....	....	[illegible]
30	....	....	....	....	....	....	....
Area, sq in	9.62	10.94	10.86	12.42	12.92	14.48	16[illegible]
I_{1-1}, in⁴.....	110	127	122	141	143	161	[illegible]
r_{1-1}, in.....	3.38	3.40	3.35	3.36	3.33	3.34	3[illegible]
I_{2-2}, in⁴.....	20.7	24.9	30.3	36.3	37.2	43.5	[illegible]
r_{2-2}, in.....	1.47	1.51	1.67	1.71	1.70	1.73	[illegible]
Weight, lb per lin ft..	32.9	37.3	37.3	42.5	44.2	49.4	[illegible]

The safe load-values above the upper heavy line are for ratios of l/r not o[ver] those between the heavy lines are for ratios up to 120 l/r; and those below the heavy line are for ratios not over 200 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

[Ta]ble XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\,l/r$$

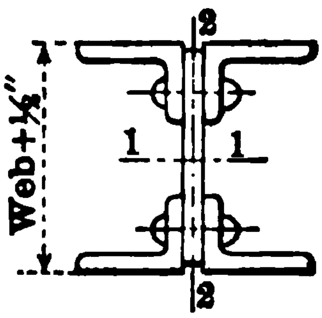
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 10″×1/4″: angles 1½″×1/4″	Web-plate 10″×1/4″: angles 2½″×1/4″	Web-plate 10″×1/4″: angles 2½″×5/16″	Web-plate 10″×5/16″: angles 3½″×5/16″	Web-plate 10″×5/16″: angles 3″×5/16″	Web-plate 10″×5/16″	Web-plate 10″×3/8″: angles 3″×3/8″	Web-plate 10″×3/8″: 4 angles 4″×3″×7/16″	Web-plate 10″×3/8″: 4 angles 5″×3½″×3/8″
								198	207
								198	207
								198	207
								192	207
								181	207
								170	203
								160	194
								149	185
								138	175
								127	166
								116	157
								106	148
								101	139
								95	130
								90	121
								84	112
								79	107
								74	103
								68	98
								63	93
								57	89
								52	84
								47	80
								...	75
								...	71
								15.23	15.9[illegible]
								267	27[illegible]
								4.19	4.1[illegible]
								43.5	70.[illegible]
								1.69	2.1[illegible]
								52.0	54.[illegible]

l/r not over 60
[b]elow the lowe[r]

[Pitts]burgh, Pa.

ıble. XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plat and-Angle Columns



Allowable fiber-stress in pounds per square inch
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\, l/r$$

Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

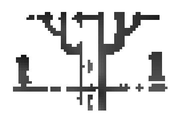
Effective length, ft	Web-plate 10"×3/8"				Web-plate 10"×1/2"			Web- 10"×
	4 angles 5"×3½"×7/16"	4 angles 6"×4"×3/8"	4 angles 6"×4"×7/16"	4 angles 6"×4"×½"	4 angles 6"×4"×½"	4 angles 6"×4"×9/16"	4 angles 6"×4"×5/8"	4 angles 6"×4"×¾"
6	232	236	266	296	312	341	370	386
7	232	236	266	296	312	341	370	386
8	232	236	266	296	312	341	370	386
9	232	236	266	296	312	341	370	386
10	232	236	266	296	312	341	370	386
11	230	236	266	296	312	341	370	386
12	220	236	266	296	312	341	370	386
13	210	235	266	296	312	341	370	386
14	200	226	257	288	302	333	363	378
15	190	218	248	278	291	321	350	365
16	180	209	238	267	280	309	337	351
17	170	201	229	257	269	297	325	338
18	160	192	220	247	258	285	312	325
19	150	184	210	237	247	274	299	312
20	140	175	201	226	236	262	287	298
21	130	167	191	216	225	250	274	285
22	123	158	182	206	214	238	261	272
23	118	150	172	195	203	226	249	258
24	113	141	163	185	192	214	236	245
25	108	132	154	175	181	203	223	232
26	103	126	144	164	170	191	210	218
27	98	121	139	157	164	181	198	207
28	93	117	134	152	158	175	192	200
29	88	113	130	146	153	169	186	193
30	83	109	125	141	147	164	179	187
rea, sq in	17.87	18.19	20.47	22.75	24.00	26.24	28.44	29.69
-1, in^4	315	319	361	401	412	451	489	50(
-1, in.	4.20	4.19	4.20	4.20	4.14	4.15	4.15	4.1(
-2, in^4	82.3	119	139	160	165	186	206	213
-2, in.	2.15	2.56	2.61	2.65	2.62	2.66	2.69	2.68
Weight, per lin ft	60.8	62.0	70.0	77.6	81.8	89.4	97.0	101.:

The safe load-values above the upper heavy line are for ratios of l/r not over 6 ose between the heavy lines are for ratios up to 120 l/r; and those below the low avy line are for ratios not over 200 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV * (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns

lowable fiber-stress in pounds per square inch:
o for lengths of 60 radii or under



Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\ l/r$$

Weights do not include rivet-heads or other details

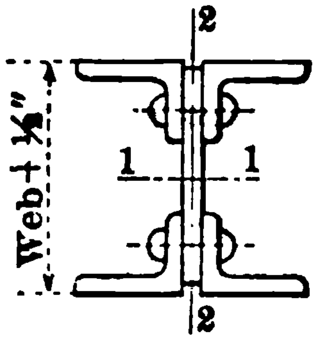
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 12"×¼"			Web-plate 12"×5/16"		Web-plate 12"×3/8"			
	4 angles 3½"×2½"×¼"	4 angles 3½"×2½"×5/16"	4 angles 4"×3"×5/16"	4 angles 4"×3"×5/16"	4 angles 4"×3"×3/8"	4 angles 4"×3"×3/8"	4 angles 5"×3½"×3/8"	4 angles 5"×3½"×7/16"	4 angles 5"×3½"×½"
6	114	132	148	157	178	187	217	242	266
7	112	132	148	157	178	187	217	242	266
8	104	123	148	157	178	187	217	242	266
9	96	115	140	147	169	177	217	242	266
10	89	106	131	138	159	167	217	242	266
11	81	98	123	129	149	156	210	237	264
12	73	89	114	120	139	145	201	226	252
13	65	80	106	111	129	134	191	215	241
14	59	72	97	101	119	124	181	205	229
15	55	67	89	92	109	113	171	194	218
16	52	63	80	84	99	102	162	184	206
17	48	58	76	79	92	96	152	173	195
18	44	54	71	75	87	91	142	162	184
19	40	50	67	70	82	85	132	152	172
20	36	45	63	65	77	80	123	141	161
21	32	41	59	61	72	75	115	130	149
22	28	37	55	56	67	69	110	125	141
23	.....	33	50	52	62	64	105	120	135
24	.....	.....	46	47	57	58	100	114	129
25	.....	.....	42	42	52	53	95	109	123
26	.....	.....	38	38	47	48	91	104	118
27	.....	.....	.....	.....	42	.....	86	98	112
28	.....	.....	.....	.....	.....	.....	81	93	106
29	.....	.....	.....	.....	.....	.....	76	88	101
30				.....			71	82	95
Area, sq in	8.76	10.12	11.36	12.11	13.67	14.42	16.70	18.62	20.50
I_{1-1}, in⁴	222	264	295	304	350	359	421	476	526
r_{1-1}, in	5.04	5.11	5.09	5.01	5.06	4.99	5.02	5.05	5.07
I_{2-2}, in⁴	16.0	20.2	29.6	30.3	36.3	37.3	70.6	82.3	94.6
r_{2-2}, in	1.35	1.41	1.61	1.58	1.63	1.61	2.06	2.10	2.15
Weight, lb per lin ft	29.8	34.6	39.0	41.6	46.8	49.1	56.9	63.3	69.7

The safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those below the lower heavy line are for ratios not over 200 l/r.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

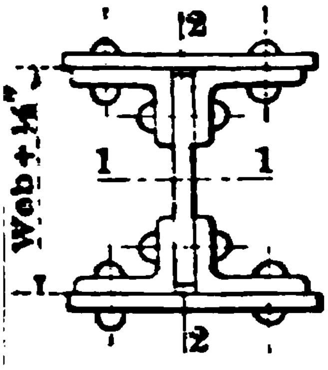
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 12″×3/8″		Web-plate 12″×1/2″					Web-plate 12″×5/8″	Web-plate 12″×3/4″
	4 angles 6″×4″×7/16″	4 angles 6″×4″×1/2″	4 angles 6″×4″×1/2″	4 angles 6″×4″×9/16″	4 angles 6″×4″×5/8″	4 angles 6″×4″×11/16″	4 angles 6″×4″×3/4″	4 angles 6″×4″×3/4″	4 angles 6″×4″×3/4″
6	276	305	325	354	383	411	439	458	478
7	276	305	325	354	383	411	439	458	478
8	276	305	325	354	383	411	439	458	478
9	276	305	325	354	383	411	439	458	478
10	276	305	325	354	383	411	439	458	478
11	276	305	325	354	383	411	439	458	478
12	276	305	325	354	383	411	439	458	478
13	274	305	323	354	383	411	439	458	478
14	264	295	312	342	373	403	433	451	469
15	254	284	300	330	359	389	418	435	452
16	244	274	288	317	346	375	403	419	436
17	234	263	277	305	333	361	388	404	420
18	224	252	265	292	319	347	373	388	403
19	214	241	253	280	306	333	358	372	387
20	204	230	242	267	293	318	344	357	370
21	194	220	230	255	279	304	329	341	354
22	184	209	218	242	266	290	314	325	338
23	174	198	207	230	253	276	299	310	321
24	164	187	195	217	239	262	284	294	305
25	155	176	183	204	226	248	269	278	288
26	147	166	173	192	213	234	254	262	272
27	142	160	167	185	203	220	239	247	256
28	137	154	162	179	196	213	230	239	248
29	132	149	156	173	189	206	223	231	240
30	127	143	150	166	183	199	215	223	232
Area, sq in	21.22	23.50	25.00	27.24	29.44	31.60	33.76	35.26	36.76
I_{1-1}, in⁴	544	605	623	683	741	794	849	867	885
r_{1-1}, in	5.06	5.07	4.99	5.01	5.02	5.01	5.01	4.96	4.91
I_{2-2}, in⁴	139	160	165	186	206	228	249	257	266
r_{2-2}, in	2.56	2.61	2.57	2.61	2.65	2.69	2.72	2.70	2.69
Weight, lb per lin ft	72.5	80.1	85.2	92.8	100.4	107.6	114.8	119.9	125.0

The safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those below the lower heavy line are for ratios not over 200 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV * (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\, l/r$$

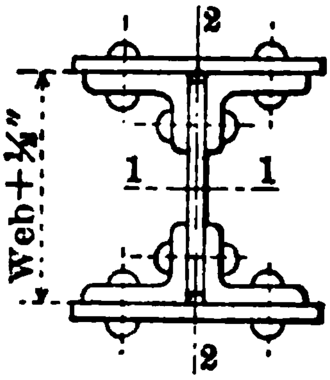
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 12"×3/8": 4 angles 6"×4"×3/8", 2 plates 14"×3/8"	Web-plate 12"×3/8": 4 angles 6"×4"×3/8", 2 plates 14"×1/2"	Web-plate 12"×3/8": 4 angles 6"×4"×7/16", 2 plates 14"×1/2"	Web-plate 12"×3/8": 4 angles 6"×4"×1/2", 2 plates 14"×1/2"	Web-plate 12"×1/2": 4 angles 6"×4"×1/2", 2 plates 14"×1/2"	Web-plate 12"×1/2": 4 angles 6"×4"×1/2", 2 plates 14"×5/8"
11	383	428	458	487	507	553
12	383	428	458	487	507	553
13	383	428	458	487	507	553
14	383	428	458	487	507	553
15	383	428	458	487	507	553
16	379	428	458	487	506	553
17	368	419	447	475	491	542
18	357	407	434	461	476	526
19	346	395	421	447	461	510
20	334	383	407	433	447	495
21	323	370	394	419	432	479
22	312	358	381	405	417	463
23	301	346	368	391	403	448
24	289	334	355	377	388	432
25	278	322	342	363	373	416
26	267	310	329	349	358	401
27	256	297	316	335	344	385
28	244	285	303	321	329	369
29	233	273	290	307	314	354
30	222	261	277	293	299	338
31	211	249	264	279	285	323
32	203	237	250	265	272	307
33	197	228	242	257	264	294
34	191	221	235	250	257	287
35	186	215	229	243	249	279
Area, sq in	29.44	32.94	35.22	37.50	39.00	42.50
I_{1-1}, in⁴	916	1 073	1 136	1 197	1 215	1 377
r_{1-1}, in	5.58	5.71	5.68	5.65	5.58	5.69
I_{2-2}, in⁴	291	348	368	388	394	451
r_{2-2}, in	3.14	3.25	3.23	3.22	3.18	3.26
Weight, per lin ft	100.2	112.1	120.1	127.7	132.8	144.7

The safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those below the lower heavy line are for ratios not over 200 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV * (Continued). Safe Loads in Units of 1 000 Pounds for Pla[illegible] and-Angle Columns



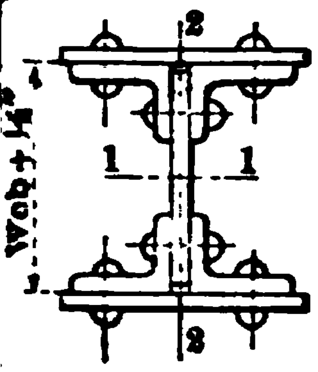
Allowable fiber-stress in pounds per square inc[illegible]
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, [illegible]
Formula (13),
$$S = 19\,000 - 100\,l/r$$
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 12"×½"		Web-plate 12"×⅝"			
	4 angles 6"×4"×9/16" 2 plates 14"×⅝"	4 angles 6"×4"×⅝" 2 plates 14"×⅝"	4 angles 6"×4"×⅝" 2 plates 14"×⅝"	4 angles 6"×4"×⅝" 2 plates 14"×¾"	4 angles 6"×4"×⅝" 2 plates 14"×⅞"	4 angles 6"×4"×⅝" 2 plates [illegible]
11	582	610	630	675	721	766
12	582	610	630	675	721	766
13	582	610	630	675	721	766
14	582	610	630	675	721	766
15	582	610	630	675	721	766
16	582	610	630	675	721	766
17	569	596	613	663	714	763
18	553	579	594	644	694	742
19	536	562	576	625	674	721
20	520	544	558	606	654	70[illegible]
21	503	527	540	587	634	67[illegible]
22	487	509	522	568	614	65[illegible]
23	470	492	504	548	594	63[illegible]
24	454	475	486	529	574	61[illegible]
25	437	457	468	510	554	59[illegible]
26	421	440	450	491	534	57[illegible]
27	404	422	431	472	514	55[illegible]
28	388	405	413	453	494	53[illegible]
29	371	388	395	434	474	51[illegible]
30	354	370	377	415	454	49[illegible]
31	338	353	359	396	434	46[illegible]
32	321	336	341	377	414	44[illegible]
33	309	323	331	361	394	42[illegible]
34	301	315	322	351	381	40[illegible]
35	293	306	313	342	371	39[illegible]
Area, sq in	44.74	46.94	48.44	51.94	55.44	58.[illegible]
I_{1-1}, in⁴	1 437	1 495	1 513	1 682	1 856	2 0[illegible]
r_{1-1}, in	5.67	5.64	5.59	5.69	5.79	5.[illegible]
I_{2-2}, in⁴	472	492	499	556	613	6[illegible]
r_{2-2}, in	3.25	3.24	3.21	3.27	3.33	3.[illegible]
Weight, lb per lin ft	152.3	159.9	165.0	176.9	188.8	200[illegible]

The safe load-values above the upper heavy line are for ratios of l/r not ove[illegible] those between the heavy lines are for ratios up to 120 l/r; and those below the lo[illegible] heavy line are for ratios not over 200 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

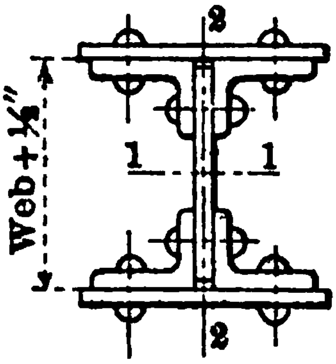
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 12"×5/8"					
	4 angles 6"×4"×5/8" 2 plates 14"×1 1/8"	4 angles 6"×4"×5/8" 2 plates 14"×1 1/4"	4 angles 6"×4"×5/8" 2 plates 14"×1 3/8"	4 angles 6"×4"×5/8" 2 plates 14"×1 1/2"	4 angles 6"×4"×5/8" 2 plates 14"×1 5/8"	4 angles 6"×4"×5/8" 2 plates 14"×1 3/4"
11	812	857	903	948	994	1 039
12	812	857	903	948	994	1 039
13	812	857	903	948	994	1 039
14	812	857	903	948	994	1 039
15	812	857	903	948	994	1 039
16	812	857	903	948	994	1 039
17	812	857	903	948	994	1 039
18	791	840	888	937	986	1 034
19	769	817	864	912	960	1 007
20	747	794	840	887	934	980
21	725	771	817	862	908	953
22	703	748	793	837	832	926
23	631	725	769	812	856	899
24	659	702	745	787	830	872
25	637	679	721	762	805	845
26	615	657	697	738	779	818
27	593	634	673	713	753	791
28	571	611	649	683	727	764
29	549	588	623	663	701	737
30	527	565	601	638	675	710
31	505	542	577	613	649	684
32	483	519	553	588	623	657
33	461	496	529	563	597	630
34	439	473	505	538	571	603
35	427	456	484	513	545	576
Area, sq in	62.44	65.94	69.44	72.94	76.44	79.94
I_{1-1}, in⁴	2 224	2 418	2 618	2 825	3 038	3 259
r_{1-1}, in	5.97	6.06	6.14	6.22	6.30	6.38
I_{2-2}, in⁴	728	785	842	899	956	1014
r_{2-2}, in	3.41	3.45	3.48	3.51	3.54	3.56
Weight, lb per lin ft	212.6	224.5	236.4	248.3	260.2	272.1

The safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those below the lower heavy line are for ratios not over 200 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\,l/r$$

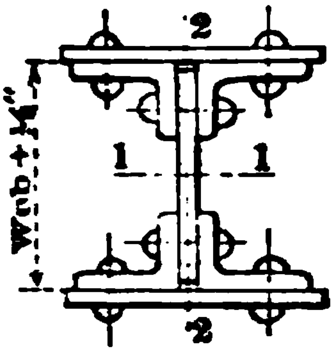
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 12"×5/8"		Web-plate 14"×3/8"				
	4 angles 6"×4"×5/8" 2 plates 14"×1 7/8"	4 angles 6"×4"×5/8" 2 plates 14"×2"	4 angles 6"×4"×3/8" 2 plates 14"×3/8"	4 angles 6"×4"×7/16" 2 plates 14"×3/8"	4 angles 6"×4"×1/2" 2 plates 14"×3/8"	4 angles 6"×4"×1/2" 2 plates 14"×7/16"	4 angles 6"×4"×1/2" 2 plates [illegible]
11	1 085	1 130	392	422	452	474	497
12	1 085	1 130	392	422	452	474	497
13	1 085	1 130	392	422	452	474	497
14	1 085	1 130	392	422	452	474	497
15	1 085	1 130	392	422	452	474	497
16	1 085	1 130	387	415	444	470	497
17	1 085	1 130	375	403	431	456	482
18	1 082	1 130	363	390	417	442	468
19	1 054	1 101	352	377	404	428	453
20	1 026	1 072	340	365	390	415	439
21	998	1 043	328	352	377	401	425
22	970	1 014	317	340	363	387	410
23	942	985	305	327	350	373	396
24	914	956	293	314	336	359	381
25	886	927	281	302	323	345	367
26	858	898	270	289	309	331	353
27	830	869	258	276	296	317	338
28	802	840	246	264	282	303	324
29	774	811	235	251	269	289	309
30	746	782	223	239	255	275	295
31	718	753	211	227	243	261	281
32	690	725	205	220	236	251	267
33	662	696	200	214	229	244	260
34	634	667	194	208	222	237	253
35	606	638	188	201	216	230	245
Area, sq in	83.44	86.94	30.19	32.47	34.75	36.50	38.[illegible]
I_{1-1}, in⁴.....	3 486	3 721	1 261	1 351	1 436	1 539	1 6[illegible]
r_{1-1}, in.....	6.46	6.54	6.46	6.45	6.43	6.49	6.[illegible]
I_{2-2}, in⁴.....	1071	1128	291	311	331	360	3[illegible]
r_{2-2}, in.....	3.58	3.60	3.10	3.09	3.09	3.14	3.[illegible]
Weight, lb per lin ft..	284.0	295.9	102.8	110.8	118.4	124.3	130[illegible]

The safe load-values above the upper heavy line are for ratios of l/r not over [illegible] those between heavy lines are for ratios up to 120 l/r; and those below the lo[illegible] heavy line are for ratios not over 200 l/r.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

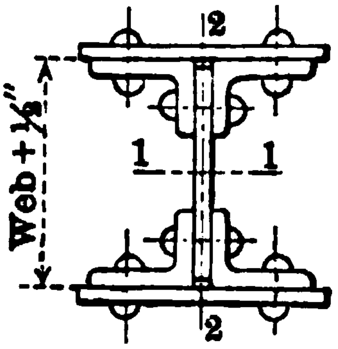
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 14″×3/8″		Web-plate 14″×1/2″		
	4 angles 6″×4″×1/2″, 2 plates 14″×9/16″	4 angles 6″×4″×1/2″, 2 plates 14″×5/8″	4 angles 6″×4″×1/2″, 2 plates 14″×5/8″	4 angles 6″×4″×9/16″, 2 plates 14″×5/8″	4 angles 6″×4″×5/8″, 2 plates 14″×5/8″
11	520	543	566	595	623
12	520	543	566	595	623
13	520	543	566	595	623
14	520	543	566	595	623
15	520	543	566	595	623
16	520	543	566	595	623
17	507	533	551	578	605
18	493	517	535	561	587
19	478	502	518	544	569
20	463	487	502	527	551
21	448	472	486	510	533
22	433	456	470	493	515
23	418	441	454	476	497
24	403	426	437	459	479
25	388	410	421	442	461
26	374	395	405	424	443
27	359	380	389	407	425
28	344	364	373	390	407
29	329	349	356	373	390
30	314	334	340	356	372
31	299	318	324	339	354
32	284	303	308	322	336
33	275	290	298	312	327
34	267	282	290	304	318
35	260	275	282	295	309
Area, sq in	40.00	41.75	43.50	45.74	47.94
I_{1-1}, in⁴	1 749	1 857	1 885	1 970	2 053
r_{1-1}, in	6.61	6.67	6.58	6.56	6.54
I_{2-2}, in⁴	417	446	451	472	492
r_{2-2}, in	3.23	3.27	3.22	3.21	3.20
Weight, lb per lin ft	136.2	142.2	148.1	155.7	163.3

The safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those below the lower heavy line are for ratios not over 200 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plat and-Angle Columns



Allowable fiber-stress in pounds per square inch: 13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\, l/r$$

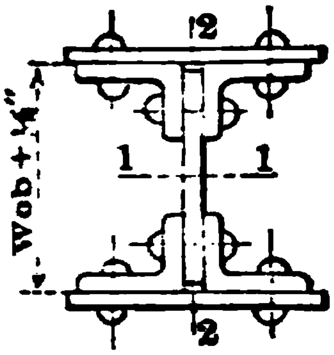
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 14"×5/8"						
	4 angles 6"×4"×5/8" 2 plates 14"×5/8"	4 angles 6"×4"×5/8" 2 plates 14"×3/4"	4 angles 6"×4"×5/8" 2 plates 14"×7/8"	4 angles 6"×4"×5/8" 2 plates 14"×1"	4 angles 6"×4"×5/8" 2 plates 14"×1 1/8"	4 angles 6"×4"×5/8" 2 plates 14"×1 1/4"	4 angles 6"×4"×5/8" 2 plates
11	646	691	737	782	828	873	919
12	646	691	737	782	828	873	919
13	646	691	737	782	828	873	919
14	646	691	737	782	828	873	919
15	646	691	737	782	828	873	919
16	643	691	737	782	828	873	919
17	624	675	726	776	826	873	919
18	606	655	705	754	803	852	901
19	587	635	684	733	780	829	876
20	568	615	664	711	758	805	851
21	549	596	643	689	735	782	827
22	530	576	622	668	713	758	802
23	511	556	602	646	690	734	778
24	493	536	581	625	667	711	753
25	474	517	560	603	645	687	728
26	455	497	540	581	622	664	704
27	436	477	519	560	600	640	679
28	417	457	498	538	577	617	655
29	399	438	477	516	554	593	630
30	380	418	457	495	532	569	605
31	361	398	436	473	509	546	581
32	345	378	415	452	487	522	556
33	336	365	396	430	464	499	532
34	326	356	385	415	444	475	507
35	317	346	375	404	432	461	489
Area, sq in	49.69	53.19	56.69	60.19	63.69	67.19	70.6
I_{1-1}, in^4.....	2 081	2 302	2 529	2 764	3 006	3 255	3 51
r_{1-1}, in.....	6.47	6.58	6.68	6.78	6.87	6.96	7.0
I_{2-2}, in^4.....	499	556	613	671	728	785	84
r_{2-2}, in.....	3.17	3.23	3.29	3.34	3.38	3.42	3.4
Weight, lb per lin ft..	169.3	181.2	193.1	205.0	216.9	228.8	240.

The safe load-values above the upper heavy line are for ratios of l/r not over 6 those between the heavy lines are for ratios up to 120 l/r; and those below t lower heavy line are for ratios not over 200 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\,l/r$$

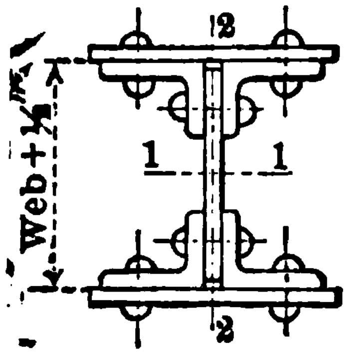
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 14"×⅝"					
	4 angles 6"×4"×⅝" 2 plates 14"×1½"	4 angles 6"×4"×⅝" 2 plates 14"×1⅝"	4 angles 6"×4"×⅝" 2 plates 14"×1¾"	4 angles 6"×4"×⅝" 2 plates 14"×1⅞"	4 angles 6"×4"×⅝" 2 plates 14"×2"	4 angles 6"×4"×⅝" 2 plates 16"×1⅞"
11	964	1 010	1 055	1 101	1 146	1 198
12	964	1 010	1 055	1 101	1 146	1 198
13	964	1 010	1 055	1 101	1 146	1 198
14	964	1 010	1 055	1 101	1 146	1 198
15	964	1 010	1 055	1 101	1 146	1 198
16	964	1 010	1 055	1 101	1 146	1 198
17	964	1 010	1 055	1 101	1 146	1 198
18	949	998	1 046	1 095	1 144	1 198
19	924	971	1 018	1 067	1 114	1 198
20	898	945	991	1 038	1 084	1 198
21	872	918	963	1 010	1 055	1 174
22	847	892	935	981	1 025	1 146
23	821	865	908	953	996	1 119
24	796	839	880	924	966	1 091
25	770	812	853	895	937	1 064
26	744	786	825	867	907	1 036
27	719	759	797	838	877	1 009
28	693	732	770	810	848	981
29	668	706	742	781	818	954
30	642	679	715	753	789	926
31	617	653	687	724	759	899
32	591	626	659	696	730	871
33	565	600	632	667	700	843
34	540	573	604	639	671	816
35	517	546	577	610	641	788
Area, sq in	74.19	77.69	81.19	84.69	88.19	92.19
I_{1-1}, in⁴	3 776	4 048	4 327	4 615	4 910	5 120
r_{1-1}, in	7.13	7.22	7.30	7.38	7.46	7.45
I_{2-2}, in⁴	899	956	1 014	1 071	1 128	1 493
r_{2-2}, in	3.48	3.51	3.53	3.56	3.58	4.02
Weight, lb per lin ft	252.6	264.5	276.4	288.3	300.2	313.8

The safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those below the lower heavy line are for ratios not over 200 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh Pa.

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\,l/r$$

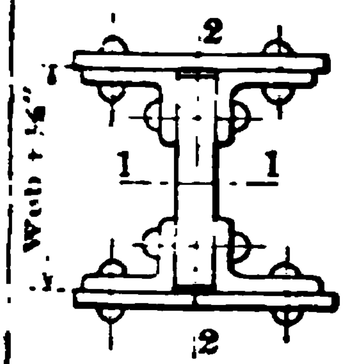
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Web-plate 14"×5/8"					
	4 angles 6"×4"×5/8" 2 plates 16"×2"	4 angles 6"×6"×5/8" 2 plates 16"×2"	4 angles 6"×6"×5/8" 2 plates 16"×2 1/8"	4 angles 6"×6"×5/8" 2 plates 16"×2 1/4"	4 angles 6"×6"×5/8" 2 plates 16"×2 3/8"	4 angles 6"×6"×5/8" 2 plates 16"×2 1/2"
11	1 250	1 315	1 367	1 419	1 471	1 523
12	1 250	1 315	1 367	1 419	1 471	1 523
13	1 250	1 315	1 367	1 419	1 471	1 523
14	1 250	1 315	1 367	1 419	1 471	1 523
15	1 250	1 135	1 367	1 419	1 471	1 523
16	1 250	1 315	1 367	1 419	1 471	1 523
17	1 250	1 315	1 367	1 419	1 471	1 523
18	1 250	1 315	1 367	1 419	1 471	1 523
19	1 250	1 315	1 367	1 419	1 471	1 523
20	1 250	1 308	1 364	1 419	1 471	1 523
21	1 229	1 277	1 333	1 388	1 443	1 497
22	1 201	1 246	1 301	1 356	1 409	1 463
23	1 172	1 216	1 269	1 323	1 375	1 428
24	1 144	1 185	1 237	1 290	1 342	1 393
25	1 115	1 154	1 206	1 258	1 308	1 359
26	1 087	1 123	1 174	1 225	1 274	1 324
27	1 058	1 093	1 142	1 192	1 241	1 289
28	1 030	1 062	1 111	1 160	1 207	1 254
29	1 001	1 031	1 079	1 127	1 173	1 220
30	973	1 000	1 047	1 094	1 139	1 185
31	944	970	1 015	1 062	1 106	1 150
32	916	939	984	1 029	1 072	1 115
33	887	908	952	996	1 038	1 081
34	859	877	920	964	1 005	1 046
35	830	847	889	931	971	1 011
Area, sq in	96.19	101.19	105.19	109.19	113.19	117.19
I_{1-1}, in⁴	5 457	5 484	5 830	6 187	6 552	6 928
r_{1-1}, in	7.53	7.36	7.44	7.53	7.61	7.69
I_{2-2}, in⁴	1 579	1 581	1 666	1 752	1 837	1 922
r_{2-2}, in	4.05	3.95	3.98	4.01	4.03	4.05
Weight, lb per lin ft	327.4	344.2	357.8	371.4	385.0	398.6

The safe load-values above the heavy line are for ratios of l/r not over 60; and those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV * (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\, l/r$$

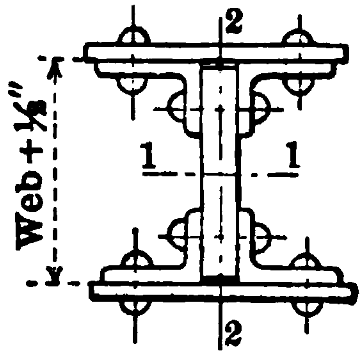
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two web-plates 14″×½″					
	4 angles 6″×6″×⅝″, 2 plates 16″×2½″	4 angles 8″×6″×⅝″, 2 plates 16″×2½″	4 angles 8″×6″×⅝″, 2 plates 18″×2⅜″	4 angles 8″×6″×⅝″, 2 plates 18″×2½″	4 angles 8″×6″×⅝″, 2 plates 18″×2⅝″	4 angles 8″×6″×⅝″, 2 plates 18″×2¾″
11	1 592	1 657	1 728	1 787	1 845	1 904
12	1 592	1 657	1 728	1 787	1 845	1 904
13	1 592	1 657	1 728	1 787	1 845	1 904
14	1 592	1 657	1 728	1 787	1 845	1 904
15	1 592	1 657	1 728	1 787	1 845	1 904
16	1 592	1 657	1 728	1 787	1 845	1 904
17	1 592	1 657	1 728	1 787	1 845	1 904
18	1 592	1 657	1 728	1 787	1 845	1 904
19	1 592	1 657	1 728	1 787	1 845	1 904
20	1 590	1 657	1 728	1 787	1 845	1 904
21	1 553	1 653	1 728	1 787	1 845	1 904
22	1 516	1 616	1 728	1 787	1 845	1 904
23	1 479	1 580	1 728	1 787	1 845	1 904
24	1 443	1 543	1 695	1 756	1 818	1 879
25	1 406	1 507	1 661	1 721	1 781	1 842
26	1 369	1 470	1 626	1 685	1 744	1 804
27	1 332	1 434	1 592	1 650	1 708	1 766
28	1 295	1 397	1 557	1 614	1 671	1 729
29	1 258	1 360	1 522	1 578	1 635	1 691
30	1 222	1 324	1 488	1 543	1 598	1 653
31	1 185	1 287	1 453	1 507	1 561	1 616
32	1 148	1 251	1 419	1 471	1 525	1 578
33	1 111	1 214	1 384	1 436	1 488	1 541
34	1 074	1 177	1 349	1 400	1 451	1 503
35	1 038	1 141	1 315	1 365	1 415	1 465
Area, sq in	122.44	127.44	132.94	137.44	141.94	146.44
I_{1-1}, in⁴	7 014	7 254	7 559	7 981	8 415	8 859
r_{1-1}, in	7.57	7.54	7.54	7.62	7.70	7.78
I_{2-2}, in⁴	1 946	2 229	2 831	2 953	3 074	3 196
r_{2-2}, in	3.99	4.18	4.61	4.63	4.65	4.67
Weight, lb per lin ft	416.4	433.6	452.3	467.6	482.9	498.2

Safe load-values above the heavy line are for ratios of l/r not over 60; those below heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXIV* (Continued). Safe Loads in Units of 1 000 Pounds for Plate-and-Angle Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

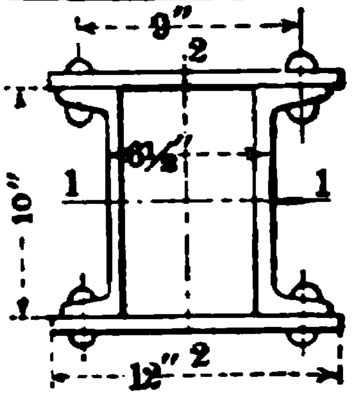
Weights do not include rivet-heads and other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two web-plates 14"×5/8"					
	4 angles 8"×6"×5/8" 2 plates 18"×2 3/4"	4 angles 8"×6"×5/8" 2 plates 20"×2 5/8"	4 angles 8"×6"×5/8" 2 plates 20"×2 3/4"	4 angles 8"×6"×5/8" 2 plates 20"×2 7/8"	4 angles 8"×6"×5/8" 2 plates 20"×3"	4 angles 8"×6"×5/8" 2 plates 20"×3 1/8"
11	1 949	2 027	2 092	2 157	2 222	2 287
12	1 949	2 027	2 092	2 157	2 222	2 287
13	1 949	2 027	2 092	2 157	2 222	2 287
14	1 949	2 027	2 092	2 157	2 222	2 287
15	1 949	2 027	2 092	2 157	2 222	2 287
16	1 949	2 027	2 092	2 157	2 222	2 287
17	1 949	2 027	2 092	2 157	2 222	2 287
18	1 949	2 027	2 092	2 157	2 222	2 287
19	1 949	2 027	2 092	2 157	2 222	2 287
20	1 949	2 027	2 092	2 157	2 222	2 287
21	1 949	2 027	2 092	2 157	2 222	2 287
22	1 949	2 027	2 092	2 157	2 222	2 287
23	1 949	2 027	2 092	2 157	2 222	2 287
24	1 918	2 027	2 092	2 157	2 222	2 287
25	1 879	2 027	2 092	2 157	2 222	2 287
26	1 841	2 009	2 077	2 146	2 214	2 283
27	1 802	1 972	2 039	2 107	2 175	2 242
28	1 763	1 935	2 002	2 068	2 135	2 202
29	1 724	1 899	1 964	2 029	2 095	2 161
30	1 686	1 862	1 926	1 991	2 055	2 120
31	1 647	1 825	1 889	1 952	2 016	2 079
32	1 608	1 789	1 851	1 913	1 976	2 039
33	1 569	1 752	1 813	1 874	1 936	1 998
34	1 530	1 715	1 775	1 836	1 896	1 957
35	1 492	1 679	1 738	1 797	1 857	1 916
Area, sq in	149.94	155.94	160.94	165.94	170.94	175.94
I_{1-1}, in⁴	8 916	9 248	9 741	10 248	10 767	11 298
r_{1-1}, in	7.71	7.70	7.78	7.86	7.94	8.01
I_{2-2}, in⁴	3 222	4 049	4 216	4 383	4 549	4 716
r_{2-2}, in	4.64	5.10	5.12	5.14	5.16	5.18
Weight, lb per lin ft	510.1	530.5	547.5	564.5	581.5	598.5

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company Pittsburgh, Pa.

Table XXV.* Safe Loads in Units of 1 000 Pounds for 10-Inch Channel-Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

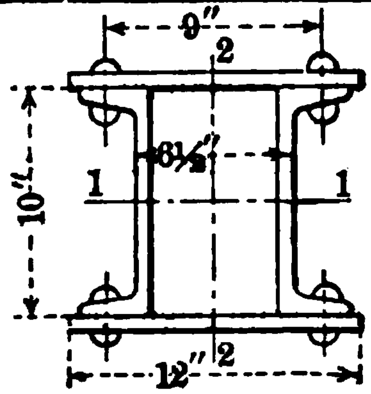
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 10-in channels latticed			Two 10-in channels, two 12-in plates					
	15.3-lb chan'ls, single lattice	20-lb channels, single lattice	25-lb channels, single lattice	15.3-lb chan'ls, 5/16-in plates	15-lb channels, 3/8-in plates	15.3-lb chan'ls, 7/16-in plates	15.3-lb chan'ls, 1/2-in plates	20-lb channels, 7/16-in plates	20-lb channels, 1/2-in plates
11	116	153	191	213	233	252	272	289	309
12	116	153	191	213	233	252	272	289	309
13	116	153	191	213	233	252	272	289	309
14	116	153	191	213	233	252	272	289	309
15	116	153	191	213	233	252	272	289	309
16	116	153	191	213	233	252	272	289	309
17	116	153	191	213	233	252	272	289	309
18	116	152	186	213	233	252	271	286	305
19	115	148	181	208	227	245	264	278	297
20	112	144	176	203	221	239	257	271	289
21	109	140	171	197	215	232	250	263	280
22	106	136	165	192	209	226	243	256	272
23	103	132	160	186	203	219	236	248	264
24	100	128	155	181	197	213	229	240	256
25	98	124	150	175	191	206	222	233	248
26	95	120	145	170	185	200	215	225	240
27	92	116	140	164	179	193	208	217	231
28	89	112	134	159	173	187	201	210	223
29	86	108	129	153	167	180	194	202	215
30	83	104	124	148	161	174	187	195	207
31	80	100	119	142	155	167	180	187	199
32	77	96	114	137	149	161	173	179	191
33	75	92	109	131	143	154	166	172	183
34	72	88	103	126	137	148	159	164	174
35	69	84	101	120	131	141	152	157	166
Area, sq in	8.92	11.76	14.70	16.42	17.92	19.42	20.92	22.26	23.76
I_{1-1}, in⁴ ...	134	158	182	333	376	420	465	444	489
r_{1-1}, in	3.87	3.66	3.52	4.30	4.58	4.65	4.71	4.46	4.53
I_{2-2}, in⁴	123	148	171	213	231	249	267	274	292
r_{2-2}, in	3.72	3.55	3.41	3.60	3.59	3.58	3.58	3.51	3.50
Weight, lb per lin ft...	37.8	47.8	57.8	55.5	60.6	65.7	70.8	75.7	80.8

Safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those below the lower heavy line are for ratios not over 200 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXV* (Continued). Safe Loads in Units of 1 000 Pounds for 10-In Channel-Columns



Allowable fiber-stress in pounds per square inch
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\, l/r$$

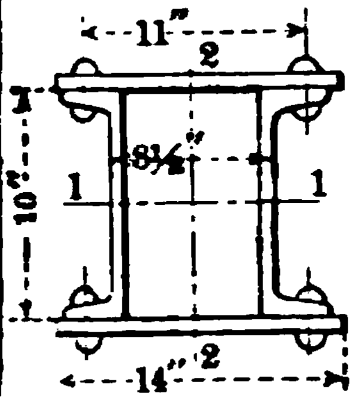
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 10-in channels, two 12-in plates							
	20-lb channels, 9/16-in plates	20-lb channels, 5/8-in plates	25-lb channels, 9/16-in plates	25-lb channels, 5/8-in plates	30-lb channels, 9/16-in plates	30-lb channels, 5/8-in plates	35-lb channels, 9/16-in plates	35-lb channels,
11	328	348	367	386	405	424	443	46
12	328	348	367	386	405	424	443	46
13	328	348	367	386	405	424	443	46
14	328	348	367	386	405	424	443	46
15	328	348	367	386	405	424	443	46
16	328	348	367	386	405	424	443	46
17	328	348	367	386	403	423	437	45
18	324	343	359	378	392	411	424	44
19	315	334	349	367	381	399	412	43
20	307	325	339	357	370	388	400	41
21	298	316	329	347	359	376	387	40
22	289	307	319	336	348	364	375	3
23	281	297	310	326	337	353	362	3
24	272	288	300	316	326	341	350	3
25	263	279	290	305	314	330	338	3
26	255	270	280	295	303	318	325	3
27	246	261	270	285	292	306	313	3
28	237	252	260	274	281	295	301	3
29	229	242	251	264	270	283	288	3
30	220	233	241	253	259	271	276	2
31	211	224	231	243	248	260	263	2
32	203	215	221	233	237	248	251	2
33	194	206	211	222	226	237	239	2
34	185	196	201	212	216	227	232	2
35	177	187	194	205	211	221	226	2
Area, sq in	25.26	26.76	28.20	29.70	31.14	32.64	34.08	35
I_{1-1}, in⁴.....	534	581	559	606	583	630	608	
r_{1-1}, in.....	4.60	4.66	4.45	4.52	4.33	4.39	4.22	4
I_{2-2}, in⁴.....	310	328	333	351	354	372	372	
r_{2-2}, in.....	3.50	3.50	3.44	3.44	3.37	3.37	3.30	3
Weight, lb per lin ft..	85.9	91.0	95.9	101.0	105.9	111.0	115.9	12

Safe load-values above the upper heavy line are for ratios of l/r not over those between the heavy lines are for ratios up to 120 l/r; and those below lower heavy line are for ratios not over 200 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXV* (Continued). Safe Loads in Units of 1 000 Pounds for 10-Inch Channel-Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\, l/r$$

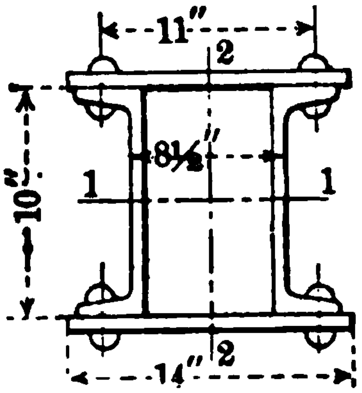
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 10-in channels, latticed				Two 10-in channels, two 14-in plates			
	15.3-lb chan'ls, single lattice	20-lb channels, single lattice	25-lb channels, single lattice	30-lb channels, single lattice	15.3-lb chan'ls, 3/8-in plates	15.3-lb chan'ls, 7/16-in plates	15.3-lb chan'ls, 1/2-in plates	20-lb channels, 7/16-in plates
11	116	153	191	229	252	275	298	312
12	116	153	191	229	252	275	298	312
13	116	153	191	229	252	275	298	312
14	116	153	191	229	252	275	298	312
15	116	153	191	229	252	275	298	312
16	116	153	191	229	252	275	298	312
17	116	153	191	229	252	275	298	312
18	116	153	189	224	252	275	298	312
19	116	150	184	218	252	275	298	312
20	114	146	179	211	252	275	298	312
21	111	142	174	205	252	275	298	312
22	109	139	169	199	251	273	295	308
23	106	135	164	193	246	267	289	302
24	103	131	159	187	241	261	282	295
25	100	127	154	180	235	256	276	288
26	98	123	149	174	230	250	270	282
27	95	119	144	168	225	244	263	275
28	92	115	139	162	219	238	257	268
29	89	112	134	156	214	232	250	261
30	87	108	129	149	209	226	244	255
31	84	104	124	143	203	220	238	248
32	81	100	119	137	198	214	231	241
33	78	96	114	131	193	209	225	235
34	75	92	109	125	187	203	219	228
35	73	88	104	121	182	197	212	221
Area, sq in	8.92	11.76	14.70	17.64	19.42	21.17	22.92	24.01
I_{1-1}, in⁴	134	158	182	207	416	468	520	491
r_{1-1}, in	3.87	3.66	3.52	3.42	4.63	4.70	4.76	4.52
I_{2-2}, in⁴	197	241	284	323	369	398	426	442
r_{2-2}, in	4.70	4.53	4.39	4.28	4.36	4.33	4.31	4.29
Weight, lb per lin ft	39.3	49.4	59.4	69.4	65.7	71.7	77.6	81.7

Safe load-values above the upper heavy line are for ratios of l/r not over 60; those between the heavy lines are for ratios up to 120 l/r; and those below the lower heavy line are for ratios not over 200 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXV* (Continued). Safe Loads in Units of 1 000 Pounds for 10-i Channel-Columns



Allowable fiber-stress in pounds per square inc
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii,
Formula (13),

$$S = 19\,000 - 100\,l/r$$

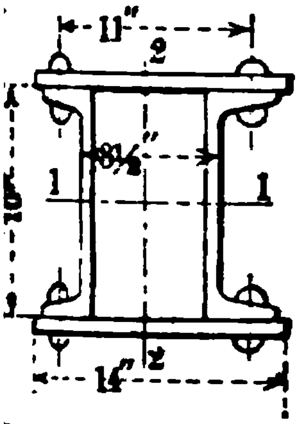
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 10-in channels, two 14-in plates						
	20-lb channels, ½-in plates	20-lb channels, 9/16-in plates	20-lb channels, ⅝-in plates	25-lb channels, 9/16-in plates	25-lb channels, ⅝-in plates	25-lb channels, 11/16-in plates	25-lb channels,
11	335	358	380	396	419	441	4
12	335	358	380	396	419	441	4
13	335	358	380	396	419	441	4
14	335	358	380	396	419	441	4
15	335	358	380	396	419	441	4
16	335	358	380	396	419	441	4
17	335	358	380	396	419	441	4
18	335	358	380	396	419	441	4
19	335	358	380	396	419	441	4
20	335	358	380	396	419	441	4
21	335	358	380	396	419	441	4
22	330	352	374	388	410	432	4
23	323	344	365	379	401	422	4
24	316	337	357	371	392	412	4
25	308	329	349	362	382	403	4
26	301	321	341	353	373	393	41
27	294	313	332	345	364	383	40
28	287	306	324	336	355	373	3
29	279	298	316	327	346	364	3
30	272	290	308	319	336	354	3
31	265	282	299	310	327	344	3
32	258	275	291	301	318	335	3
33	251	267	283	293	309	325	3
34	243	259	274	284	300	315	3
35	236	251	266	275	291	306	3
Area, sq in	25.76	27.51	29.26	30.45	32.20	33.95	35.
I_{1-1}, in^4	544	597	652	622	676	732	7
r_{1-1}, in	4.59	4.66	4.72	4.52	4.58	4.64	4.
I_{2-2}, in^4	470	499	527	541	570	598	[illegible]
r_{2-2}, in	4.27	4.26	4.24	4.22	4.21	4.20	4.
Weight, lb per lin ft	87.6	93.6	99.5	103.6	109.5	115.5	121

Safe load-values above the heavy line are for ratios of l/r not over 60; and th below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXV* (Continued). Safe Loads in Units of 1 000 Pounds for 10-Inch Channel-Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\,l/r$$

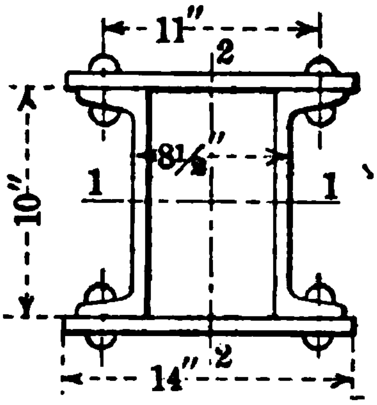
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 10-in channels, two 14-in plates					
	30-lb channels, 11/16-in plates	30-lb channels, ¾-in plates	30-lb channels, 13/16-in plates	30-lb channels, ⅞-in plates	30-lb channels, 15/16-in plates	30-lb channels, 1-in plates
11	480	502	525	548	571	593
12	480	502	525	548	571	593
13	480	502	525	548	571	593
14	480	502	525	548	571	593
15	480	502	525	548	571	593
16	480	502	525	548	571	593
17	480	502	525	548	571	593
18	480	502	525	548	571	593
19	480	502	525	548	571	593
20	480	502	525	548	571	593
21	477	500	522	544	567	589
22	467	488	510	532	554	575
23	456	477	499	520	541	562
24	446	466	487	508	529	549
25	435	455	475	495	516	536
26	424	444	464	483	503	522
27	414	432	452	471	490	509
28	403	421	440	459	478	496
29	392	410	429	446	465	483
30	382	399	417	434	452	469
31	371	388	405	422	440	456
32	360	377	394	410	427	443
33	350	365	382	398	414	430
34	339	354	370	385	401	416
35	328	343	359	373	389	403
Area, sq in	36.89	38.64	40.39	42.14	43.89	45.64
I_{1-1}, in^4	757	814	873	932	994	1 056
r_{1-1}, in	4.53	4.59	4.65	4.70	4.76	4.81
I_{2-2}, in^4	637	666	695	723	752	780
r_{2-2}, in	4.16	4.15	4.15	4.14	4.14	4.13
Weight, per lin ft	125.5	131.4	137.4	143.3	149.3	155.2

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXV* (Continued). Safe Loads in Units of 1 000 Pounds for 10-In Channel-Columns



Allowable fiber-stress in pounds per square inch
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\, l/r$$

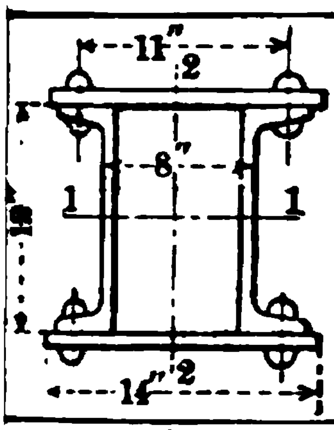
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 10-in channels, two 14-in plates					
	35-lb channels, 15/16-in plates	35-lb channels, 1-in plates	35-lb channels, 1 1/16-in plates	35-lb channels, 1 1/8-in plates	35-lb channels, 1 3/16-in plates	35-lb channels, 1¼-in plates
11	609	632	654	677	700	723
12	609	632	654	677	700	723
13	609	632	654	677	700	723
14	609	632	654	677	700	723
15	609	632	654	677	700	723
16	609	632	654	677	700	723
17	609	632	654	677	700	723
18	609	632	654	677	700	723
19	609	632	654	677	700	723
20	609	632	654	677	700	723
21	602	624	647	669	691	714
22	588	610	632	654	675	697
23	575	596	617	639	660	681
24	561	582	603	624	644	665
25	547	568	588	608	628	648
26	533	553	573	593	612	632
27	520	539	559	578	596	616
28	506	525	544	563	581	599
29	492	511	529	547	565	583
30	479	496	514	532	549	567
31	465	482	500	517	533	550
32	451	468	485	502	517	534
33	437	454	470	487	502	518
34	424	440	455	471	486	502
35	410	425	441	456	470	485
Area, sq in	46.83	48.58	50.33	52.08	53.83	55.5
I_{1-1}, in⁴	1 018	1 080	1 144	1 209	1 275	1 34
r_{1-1}, in	4.66	4.72	4.77	4.82	4.87	4.9
I_{2-2}, in⁴	788	816	845	874	902	93
r_{2-2}, in	4.10	4.10	4.10	4.10	4.09	4.0
Weight, lb per lin ft	159.3	165.2	171.2	177.1	183.1	189.

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVI.* Safe Loads in Units of 1 000 Pounds for 12-Inch Channel-Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\, l/r$$

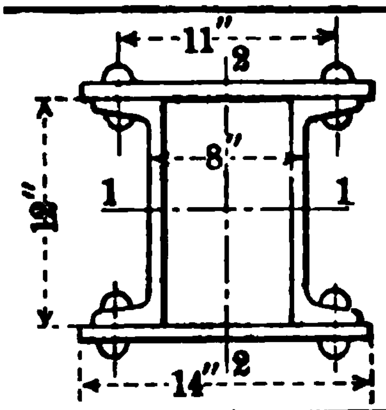
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 12-in channels, latticed				Two 12-in channels, two 14-in plates		
	20.7-lb chan'ls, single lattice	25-lb channels, single lattice	30-lb channels, single lattice	35-lb channels, single lattice	20.7-lb chan'ls, 3/8-in plates	20.7-lb chan'ls, 7/16-in plates	20.7-lb chan'ls, 1/2-in plates
11	157	191	229	268	293	316	339
12	157	191	229	268	293	316	339
13	157	191	229	268	293	316	339
14	157	191	229	268	293	316	339
15	157	191	229	268	293	316	339
16	157	191	229	268	293	316	339
17	157	191	229	268	293	316	339
18	157	191	229	268	293	316	339
19	157	191	229	268	293	316	339
20	157	191	229	268	293	316	339
21	157	191	229	265	293	316	339
22	157	190	225	259	290	312	334
23	155	186	220	253	283	305	326
24	152	182	215	248	277	298	319
25	149	178	210	242	271	291	312
26	146	174	205	236	265	284	304
27	142	170	200	230	258	277	297
28	139	166	195	224	252	271	290
29	136	162	190	218	246	264	282
30	133	158	185	212	239	257	275
31	129	154	180	206	233	250	268
32	126	150	175	200	227	243	260
33	123	146	170	194	220	236	253
34	120	142	165	188	214	230	246
35	117	138	160	182	208	223	238
Area, sq in	12.06	14.70	17.64	20.58	22.56	24.31	26.06
I_{1-1}, in⁴	256	288	323	359	658	730	803
r_{1-1}, in	4.61	4.43	4.28	4.17	5.40	5.48	5.55
I_{2-2}, in⁴	244	279	316	351	415	444	473
r_{2-2}, in	4.50	4.36	4.23	4.13	4.29	4.27	4.26
Weight, per lin ft	50.4	59.4	69.4	79.4	76.7	82.7	88.6

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVI* (Continued). Safe Loads in Units of 1 000 Pounds for 12-in Channel-Columns



Allowable fiber-stress in pounds per square inch
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

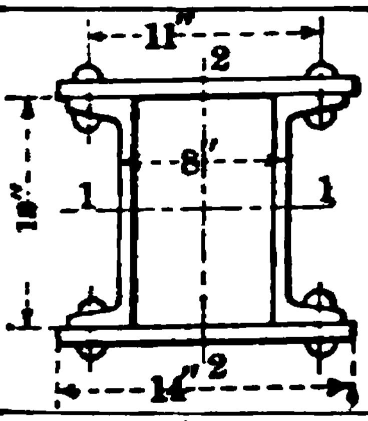
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 12-in channels, two 14-in plates						
	20.7-lb chan'ls, 9/16-in plates	20.7-lb chan'ls, 5/8-in plates	25-lb channels, 9/16-in plates	25-lb channels, 5/8-in plates	25-lb channels, 11/16-in plates	25-lb channels, 3/4-in plates	25-lb channels, 13/16-in plates
11	362	384	396	419	441	464	487
12	362	384	396	419	441	464	487
13	362	384	396	419	441	464	487
14	362	384	396	419	441	464	487
15	362	384	396	419	441	464	487
16	362	384	396	419	441	464	487
17	362	384	396	419	441	464	487
18	362	384	396	419	441	464	487
19	362	384	396	419	441	464	487
20	362	384	396	419	441	464	487
21	362	384	396	418	440	463	485
22	355	377	387	409	431	453	474
23	347	369	378	400	421	443	464
24	339	360	370	390	411	432	453
25	332	352	361	381	401	422	44
26	324	344	352	372	392	412	43
27	316	335	344	363	382	402	42
28	308	327	335	354	372	391	41
29	300	318	326	344	362	381	39
30	292	310	318	335	353	371	38
31	284	302	309	326	343	361	37
32	277	293	300	317	333	350	36
33	269	285	291	307	323	340	35
34	261	277	283	298	314	330	34
35	253	268	274	289	304	320	33
Area, sq in	27.81	29.56	30.45	32.20	33.95	35.70	37.4
I_{1-1}, in⁴	878	954	910	986	1 063	1 142	1 2
r_{1-1}, in	5.62	5.68	5.47	5.53	5.60	5.66	5.
I_{2-2}, in⁴	501	530	537	565	594	622	6
r_{2-2}, in	4.24	4.23	4.20	4.19	4.18	4.18	4.
Weight, lb per lin ft	94.6	100.5	103.6	109.5	115.5	121.4	127

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVI* (Continued). Safe Loads in Units of 1 000 Pounds for 12-Inch Channel-Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\, l/r$$

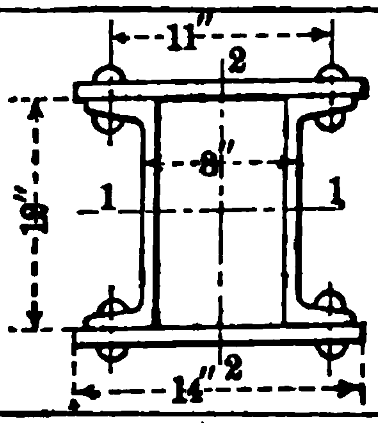
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 12-in channels, two 14-in plates							
	30-lb channels, 3/4-in plates	30-lb channels, 13/16-in plates	30-lb channels, 7/8-in plates	30-lb channels, 15/16-in plates	30-lb channels, 1-in plates	35-lb channels, 15/16-in plates	35-lb channel, 1-in plates	35-lb channels, 1 1/16-in plates
11	502	525	548	571	593	609	632	654
12	502	525	548	571	593	609	632	654
13	502	525	548	571	593	609	632	654
14	502	525	548	571	593	609	632	654
15	502	525	548	571	593	609	632	654
16	502	525	548	571	593	609	632	654
17	502	525	548	571	593	609	632	654
18	502	525	548	571	593	609	632	654
19	502	525	548	571	593	609	632	654
20	502	525	548	571	593	609	632	654
21	498	521	543	565	588	601	623	645
22	487	509	531	553	575	587	609	631
23	476	497	518	540	561	573	594	616
24	465	486	506	527	548	559	580	601
25	453	474	494	514	535	545	566	586
26	442	462	482	502	522	532	552	571
27	431	451	469	489	508	518	537	557
28	420	439	457	476	495	504	523	542
29	409	427	445	463	482	490	509	527
30	397	415	432	450	468	477	494	512
31	386	404	420	438	455	463	480	497
32	375	392	408	425	442	449	466	483
33	364	380	396	412	428	435	452	468
34	352	368	383	399	415	421	437	453
35	341	357	371	386	402	408	423	438
Area, sq in	38.64	40.39	42.14	43.89	45.64	46.83	48.58	50.33
I_{1-1}, in^4	1 174	1 258	1 340	1 424	1 509	1 459	1 544	1 630
r_{1-1}, in	5.52	5.58	5.64	5.70	5.75	5.58	5.64	5.69
I_{2-2}, in^4	659	688	717	745	774	779	808	837
r_{2-2}, in	4.13	4.13	4.12	4.12	4.12	4.08	4.08	4.08
Weight, per lin ft	131.4	137.4	143.3	149.3	155.2	159.3	165.2	171.2

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVI* (Continued). Safe Loads in Units of 1 000 Pounds for 12-I Channel-Columns



Allowable fiber-stress in pounds per square inch
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, b
Formula (13).

$$S = 19\,000 - 100\,l/r$$

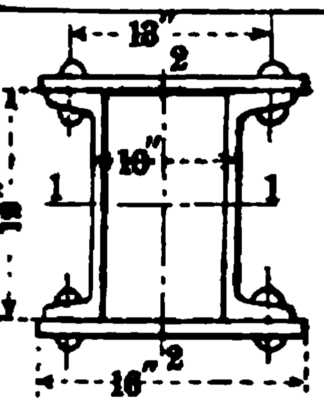
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 12-in channels, two 14-in plates						
	35-lb channels, 1⅛-in plates	35-lb channels, 1 3/16-in plates	35-lb channels, 1¼-in plates	35-lb channels, 1 5/16-in plates	35-lb channels, 1⅜-in plates	35-lb channels, 1 7/16-in plates	35-lb channels, 1
11	677	700	723	745	768	791	81
12	677	700	723	745	768	791	81
13	677	700	723	745	768	791	81
14	677	700	723	745	768	791	81
15	677	700	723	745	768	791	81
16	677	700	723	745	768	791	81
17	677	700	723	745	768	791	81
18	677	700	723	745	768	791	81
19	677	700	723	745	768	791	81
20	677	700	723	745	768	791	81
21	668	689	712	734	757	779	80
22	653	674	695	717	739	761	78
23	637	658	679	700	722	743	76
24	622	642	663	684	704	725	74
25	607	626	646	667	687	707	72
26	591	610	630	650	670	689	70
27	576	594	614	633	652	672	69
28	561	578	597	616	635	654	67
29	545	563	581	599	617	636	65
30	530	547	564	582	600	618	63
31	515	531	548	565	583	600	61
32	499	515	532	548	565	582	59
33	484	499	515	531	548	564	58
34	469	483	499	515	530	546	56
35	453	467	482	498	513	528	54
Area, sq in	52.08	53.83	55.58	57.33	59.08	60.83	62.
I_{1-1}, in⁴	1 719	1 808	1 899	1 992	2 087	2 183	2 2
r_{1-1}, in.	5.74	5.80	5.85	5.89	5.94	5.99	6.
I_{2-2}, in⁴	865	894	922	951	980	1 008	1 0
r_{2-2}, in.	4.08	4.07	4.07	4.07	4.07	4.07	4.
Weight, lb per lin ft.	177.1	183.1	189.0	195.0	200.9	206.9	212

Safe load-values above the heavy line are for ratios of l/r not over 60; th below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVI* (Continued). Safe Loads in Units of 1 000 Pounds for 12-Inch Channel-Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

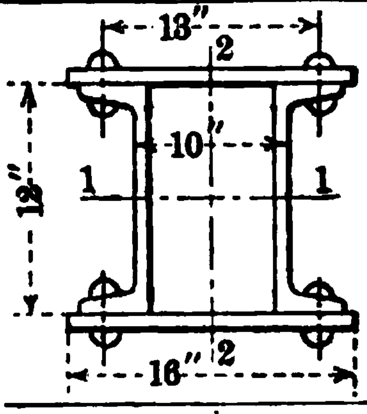
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 12-in channels, two 16-in plates									
	30-lb channels, 15/16-in plates	30-lb channels, 1-in plates	30-lb channels, 1 1/16-in plates	30-lb channels, 1 1/8-in plates	30-lb channels, 1 3/16-in plates	30-lb channels, 1 1/4-in plates	35-lb channels, 1 3/16-in plates	35-lb channels, 1 1/4-in plates	35-lb channels, 1 5/16-in plates	35-lb channels, 1 3/8-in plates
11	619	645	671	697	723	749	762	788	814	840
12	619	645	671	697	723	749	762	788	814	840
13	619	645	671	697	723	749	762	788	814	840
14	619	645	671	697	723	749	762	788	814	840
15	619	645	671	697	723	749	762	788	814	840
16	619	645	671	697	723	749	762	788	814	840
17	619	645	671	697	723	749	762	788	814	840
18	619	645	671	697	723	749	762	788	814	840
19	619	645	671	697	723	749	762	788	814	840
20	619	645	671	697	723	749	762	788	814	840
21	619	645	671	697	723	749	762	788	814	840
22	619	645	671	697	723	749	762	788	814	840
23	619	645	671	697	723	749	762	788	814	840
24	619	645	671	697	723	749	762	787	813	838
25	610	635	660	686	711	736	747	772	797	822
26	599	623	648	673	697	721	732	756	781	805
27	587	611	635	659	683	707	718	741	766	789
28	575	599	622	646	669	693	703	726	750	773
29	563	586	609	633	655	678	688	711	734	757
30	552	574	596	619	642	664	674	696	719	741
31	540	562	583	606	628	649	659	681	703	724
32	528	549	571	593	614	635	644	665	687	708
33	516	537	558	579	600	621	630	650	672	692
34	504	525	545	566	586	606	615	635	656	676
35	493	512	532	553	572	592	600	620	640	660
Area, sq in	47.64	49.64	51.64	53.64	55.64	57.64	58.58	60.58	62.58	64.58
I_{1-1}, in⁴	1 581	1 678	1 777	1 878	1 980	2 084	2 015	2 119	2 225	2 333
r_{1-1}, in	5.76	5.81	5.87	5.92	5.97	6.01	5.87	5.91	5.96	6.01
I_{2-2}, in⁴	1 121	1 164	1 206	1 249	1 292	1 334	1 349	1 392	1 434	1 477
r_{2-2}, in	4.85	4.84	4.83	4.83	4.82	4.81	4.80	4.79	4.79	4.78
Weight, per lin ft	162.0	168.8	175.6	182.4	189.2	196.0	199.2	206.0	212.8	219.6

The load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

*From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVI * (Continued). Safe Loads in Units of 1 000 Pounds for 12-in Channel-Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\, l/r$$

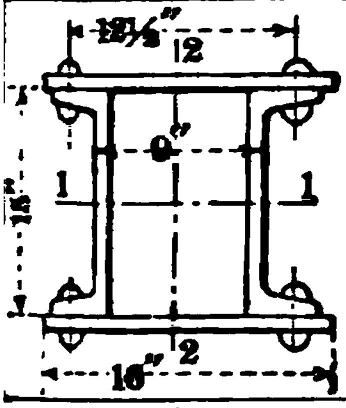
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 12-in channels, two 16-in plates									
	35-lb channels, 1 7/16-in plates	35-lb channels, 1 1/2-in plates	35-lb channels, 1 9/16-in plates	35-lb channels, 1 5/8-in plates	35-lb channels, 1 11/16-in plates	35-lb channels, 1 3/4-in plates	35-lb channels, 1 13/16-in plates	35-lb channels, 1 7/8-in plates	35-lb channels, 1 15/16-in plates	35-lb channels,
11	866	892	918	944	970	996	1 022	1 048	1 074	1
12	866	892	918	944	970	996	1 022	1 048	1 074	1
13	866	892	918	944	970	996	1 022	1 048	1 074	1
14	866	892	918	944	970	996	1 022	1 048	1 074	1
15	866	892	918	944	970	996	1 022	1 048	1 074	1
16	866	892	918	944	970	996	1 022	1 048	1 074	1
17	866	892	918	944	970	996	1 022	1 048	1 074	1
18	866	892	918	944	970	996	1 022	1 048	1 074	1
19	866	892	918	944	970	996	1 022	1 048	1 074	1
20	866	892	918	944	970	996	1 022	1 048	1 074	1
21	866	892	918	944	970	996	1 022	1 048	1 074	1
22	866	892	918	944	970	996	1 022	1 048	1 074	1
23	866	892	918	944	970	996	1 022	1 048	1 074	1
24	864	889	915	940	966	992	1 017	1 042	1 068	1
25	847	872	897	922	947	972	997	1 022	1 047	1
26	830	854	879	903	928	953	977	1 002	1 027	1
27	814	837	862	885	909	934	957	981	1 006	1
28	797	820	844	867	891	914	937	961	985	1
29	780	803	826	848	872	895	917	941	964	
30	764	785	808	830	853	876	897	920	943	
31	747	768	791	812	834	857	878	900	922	
32	730	751	773	794	815	837	858	880	901	
33	713	734	755	775	797	818	838	859	881	
34	697	716	737	757	778	799	818	839	860	
35	680	699	720	739	759	779	798	819	839	
Area, sq in	66.58	68.58	70.58	72.58	74.58	76.58	78.58	80.58	82.58	8
I_{1-1}, in⁴.....	2 443	2 555	2 668	2 783	2 901	3 020	3 141	3 264	3 389	
r_{1-1}, in.....	6.06	6.10	6.15	6.19	6.24	6.28	6.32	6.36	6.41	
I_{2-2}, in⁴.....	1 520	1 562	1 605	1 648	1 690	1 733	1 776	1 818	1 861	1
r_{2-2}, in.....	4.78	4.77	4.77	4.76	4.76	4.76	4.75	4.75	4.75	
Weight, lb per lin ft..	226.4	233.2	240.0	246.8	253.6	260.4	267.2	274.0	280.8	

Safe load-values above the heavy line are for ratios of l/r not over 60; th below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVII.* Safe Loads in Units of 1 000 Pounds for 15-Inch Channel-Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\, l/r$$

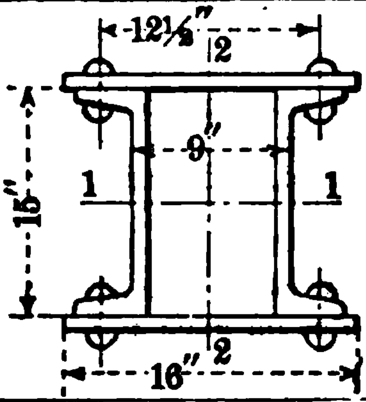
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 15-in channels, latticed: 33.9-lb chan'ls, single lattice	35-lb channels, single lattice	40-lb channels, single lattice	45-lb channels, single lattice	Two 15-in channels, two 16-in plates: 33.9-lb chan'ls, 3/8-in plates	33.9-lb chan'ls, 7/16-in plates	33.9-lb chan'ls, 1/2-in plates
11	257	268	306	344	413	439	465
12	257	268	306	344	413	439	465
13	257	268	306	344	413	439	465
14	257	268	306	344	413	439	465
15	257	268	306	344	413	439	465
16	257	268	306	344	413	439	465
17	257	268	306	344	413	439	465
18	257	268	306	344	413	439	465
19	257	268	306	344	413	439	465
20	257	268	306	344	413	439	465
21	257	268	306	344	413	439	465
22	257	268	306	344	413	439	465
23	257	268	306	344	413	439	465
24	257	268	306	343	413	439	465
25	257	266	301	336	407	432	457
26	252	261	295	329	400	424	448
27	247	256	289	322	392	415	440
28	243	251	284	316	384	407	431
29	238	246	278	309	376	399	422
30	233	241	272	302	368	390	413
31	228	236	266	296	360	382	404
32	224	231	260	289	352	373	395
33	219	226	254	282	345	365	386
34	214	221	249	276	337	357	377
35	209	216	243	269	329	348	368
Area, sq in	19.80	20.58	23.52	26.48	31.80	33.80	35.80
I_{1-1}, in⁴	625	640	695	750	1 334	1 459	1 586
r_{1-1}, in	5.62	5.58	5.43	5.32	6.48	6.57	6.66
I_{2-2}, in⁴	491	504	552	597	747	789	832
r_{2-2}, in	4.98	4.95	4.84	4.75	4.85	4.83	4.82
Weight, per lin ft	80.2	84.2	92.1	102.2	106.8	113.6	120.4

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVII* (Continued). Safe Loads in Units of 1 000 Pounds for 15-in Channel-Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\, l/r$$

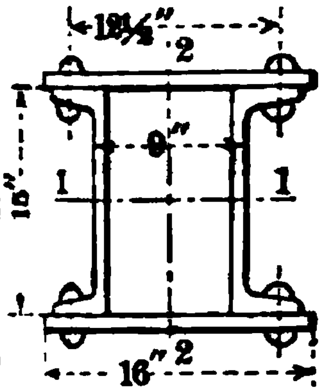
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 15-in channels, two 16-in plates						
	33.9-lb chan'ls, 9/16-in plates	33.9-lb chan'ls, 5/8-in plates	35-lb channels, 5/8-in plates	35-lb channels, 11/16-in plates	35-lb channels, 3/4-in plates	35-lb channels, 13/16-in plates	35-lb channels, 7/8-in plates
11	491	517	528	554	580	606	632
12	491	517	528	554	580	606	632
13	491	517	528	554	580	606	632
14	491	517	528	554	580	606	632
15	491	517	528	554	580	606	632
16	491	517	528	554	580	606	632
17	491	517	528	554	580	606	632
18	491	517	528	554	580	606	632
19	491	517	528	554	580	606	632
20	491	517	528	554	580	606	632
21	491	517	528	554	580	606	632
22	491	517	528	554	580	606	632
23	491	517	528	554	580	606	63[illegible]
24	491	517	527	552	578	604	62[illegible]
25	482	507	517	542	567	592	61[illegible]
26	473	498	507	531	555	580	60[illegible]
27	464	488	497	520	544	569	59[illegible]
28	454	478	486	510	533	557	58[illegible]
29	445	468	476	499	522	545	56[illegible]
30	435	458	466	488	511	533	55[illegible]
31	426	448	456	478	499	522	54[illegible]
32	416	438	446	467	488	510	53[illegible]
33	407	428	436	456	477	498	51[illegible]
34	398	418	425	446	466	487	50[illegible]
35	388	408	415	435	454	475	49[illegible]
Area, sq in	37.80	39.80	40.58	42.58	44.58	46.58	48.[illegible]
I_{1-1}, in^4	1 715	1 847	1 861	1 994	2 129	2 267	2 4[illegible]
r_{1-1}, in	6.74	6.81	6.77	6.84	6.91	6.98	7.[illegible]
I_{2-2}, in^4	875	917	930	973	1 016	1 058	1 [illegible]
r_{2-2}, in	4.81	4.80	4.79	4.78	4.77	4.77	4.[illegible]
Weight, lb per lin ft	127.2	134.0	138.0	144.8	151.6	158.4	16[illegible]

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVII* (Continued). Safe Loads in Units of 1 000 Pounds for 15-In Channel-Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

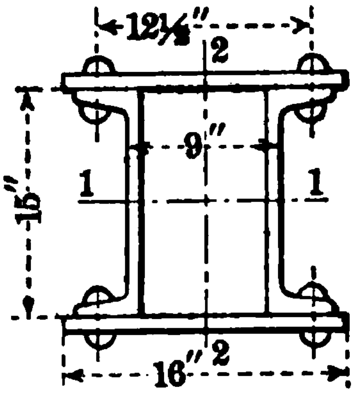
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 15-in channels, two 16-in plates						
	40-lb channels, 13/16-in plates	40-lb channels, 7/8-in plates	40-lb channels, 15/16-in plates	40-lb channels, 1-in plates	40-lb channels, 1 1/16-in plates	40-lb channels, 1 1/8-in plates	45-lb channels, 1 1/16-in plates
11	644	670	696	722	748	774	786
12	644	670	696	722	748	774	786
13	644	670	696	722	748	774	786
14	644	670	696	722	748	774	786
15	644	670	696	722	748	774	786
16	644	670	696	722	748	774	786
17	644	670	696	722	748	774	786
18	644	670	696	722	748	774	786
19	644	670	696	722	748	774	786
20	644	670	696	722	748	774	786
21	644	670	696	722	748	774	786
22	644	670	696	722	748	774	786
23	644	670	696	722	748	774	786
24	639	665	690	715	741	767	777
25	627	651	677	701	727	752	761
26	614	638	663	687	712	737	746
27	602	625	649	673	697	721	730
28	589	612	636	659	683	706	715
29	577	599	622	645	668	691	699
30	564	586	609	631	653	676	684
31	551	573	595	616	639	661	668
32	539	560	581	602	624	646	653
33	526	547	568	588	609	630	637
34	514	534	554	574	595	615	622
35	501	520	541	560	580	600	606
Area, sq in	49.52	51.52	53.52	55.52	57.52	59.52	60.48
I_{1-1}, in⁴	2 322	2 461	2 602	2 746	2 891	3 039	2 946
r_{1-1}, in	6.85	6.91	6.97	7.03	7.09	7.15	6.98
I_{2-2}, in⁴	1 106	1 149	1 192	1 234	1 277	1 320	1 322
r_{2-2}, in	4.73	4.72	4.72	4.71	4.71	4.71	4.68
Weight, lb per lin ft	168.4	175.2	182.0	188.8	195.6	202.4	205.6

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVII*. (Continued). Safe Loads in Units of 1 000 Pounds for 15-In Channel-Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\,l/r$$

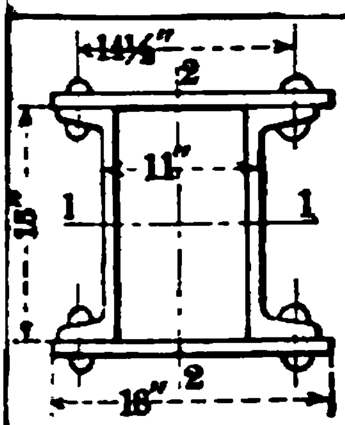
Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 15-in channels, two 16-in plates						
	45-lb channels, 1⅛-in plates	45-lb channels, 1³⁄₁₆-in plates	45-lb channels, 1¼-in plates	45-lb channels, 1⁵⁄₁₆-in plates	45-lb channels, 1⅜-in plates	45-lb channels, 1⁷⁄₁₆-in plates	45-lb channels, 1½-in plates
11	812	838	864	890	916	942	968
12	812	838	864	890	916	942	968
13	812	838	864	890	916	942	968
14	812	838	864	890	916	942	968
15	812	838	864	890	916	942	968
16	812	838	864	890	916	942	968
17	812	838	864	890	916	942	968
18	812	838	864	890	916	942	968
19	812	838	864	890	916	942	968
20	812	838	864	890	916	942	968
21	812	838	864	890	916	942	968
22	812	838	864	890	916	942	968
23	812	838	864	890	916	942	968
24	802	827	853	879	904	930	954
25	786	811	836	861	886	912	937
26	770	794	819	844	868	893	918
27	754	778	802	826	850	874	898
28	738	761	785	808	832	856	879
29	722	745	768	791	814	837	860
30	705	728	751	773	796	818	841
31	689	711	734	756	778	800	822
32	673	695	716	738	760	781	803
33	657	678	699	720	741	763	784
34	641	662	682	703	723	744	764
35	625	645	665	685	705	725	745
Area, sq in	62.48	64.48	66.48	68.48	70.48	72.48	74.4
I_{1-1}, in⁴	3 094	3 244	3 396	3 550	3 707	3 865	4 0
r_{1-1}, in	7.04	7.09	7.15	7.20	7.25	7.30	7.3
I_{2-2}, in⁴	1 365	1 408	1 450	1 493	1 536	1 578	1 6
r_{2-2}, in	4.67	4.67	4.67	4.67	4.67	4.67	4.6
Weight, lb per lin ft	212.4	219.2	226.0	232.8	239.6	246.4	253.

Safe load-values above the heavy line are for ratios of l/r not over 60; the below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVII * (Continued). Safe Loads in Units of 1 000 Pounds for 15-Inch Channel-Columns



Allowable fiber-stress in pounds per square inch
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\, l/r$$

Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 15-in channels, two 18-in plates							
	33.9-lb chan'ls, ⅜-in plates	33.9-lb chan'ls, 7/16-in plates	33.9-lb chan'ls, ½-in plates	33.9-lb chan'ls, 9/16-in plates	33.9-lb chan'ls, ⅝-in plates	35-lb channels, ⅝-in plates	35-lb channels, 11/16-in plates	35-lb channels, ¾-in plates
11	433	462	491	521	550	560	589	619
12	433	462	491	521	550	560	589	619
13	433	462	491	521	550	560	589	619
14	433	462	491	521	550	560	589	619
15	433	462	491	521	550	560	589	619
16	433	462	491	521	550	560	589	619
17	433	462	491	521	550	560	589	619
18	433	462	491	521	550	560	589	619
19	433	462	491	521	550	560	589	619
20	433	462	491	521	550	560	589	619
21	433	462	491	521	550	560	589	619
22	433	462	491	521	550	560	589	619
23	433	462	491	521	550	560	589	619
24	433	462	491	521	550	560	589	619
25	433	462	491	521	550	560	589	619
26	433	462	491	521	550	560	589	619
27	433	462	491	521	550	560	589	619
28	433	462	491	520	549	558	586	615
29	428	456	484	512	539	549	577	605
30	421	449	476	503	530	540	567	594
31	414	441	468	494	521	530	557	584
32	407	433	459	486	512	521	547	574
33	400	426	451	477	503	512	537	563
34	393	418	443	469	494	502	527	553
35	386	411	435	460	485	493	518	543
Area, sq in	33.30	35.55	37.80	40.05	42.30	43.08	45.33	47.58
I_{1-1}, in⁴	1 423	1 564	1 707	1 852	1 999	2 014	2 164	2 316
r_{1-1}, in	6.54	6.63	6.72	6.80	6.87	6.84	6.91	6.98
I_{2-2}, in⁴	1 069	1 130	1 190	1 251	1 312	1 332	1 393	1 453
r_{2-2}, in	5.67	5.64	5.61	5.59	5.57	5.56	5.54	5.53
Weight, lb per lin ft	111.9	119.6	127.2	134.9	142.5	146.5	154.2	161.8

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

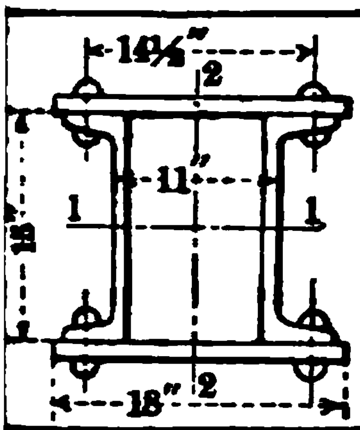
Table XXVII* (Continued). Safe Loads in Units of 1 000 Pounds for 15-in Channel-Columns

Effective length, ft	Two 15-in channels, two 18-in plates							
	35-lb channels, 13/16-in plates	35-lb channels, 7/8-in plates	40-lb channels, 13/16-in plates	40-lb channels, 7/8-in plates	40-lb channels, 15/16-in plates	40-lb channels, 1-in plates	40-lb channels, 1 1/16-in plates	40-lb channels, 1 1/8-in plates
11	648	677	686	715	745	774	803	83
12	648	677	686	715	745	774	803	83
13	648	677	686	715	745	774	803	83
14	648	677	686	715	745	774	803	83
15	648	677	686	715	745	774	803	83
16	648	677	686	715	745	774	803	83
17	648	677	686	715	745	774	803	83
18	648	677	686	715	745	774	803	83
19	648	677	686	715	745	774	803	83
20	648	677	686	715	745	774	803	83
21	648	677	686	715	745	774	803	83
22	648	677	686	715	745	774	803	83
23	648	677	686	715	745	774	803	83
24	648	677	686	715	745	774	803	83
25	648	677	686	715	745	774	803	83
26	648	677	686	715	745	774	803	83
27	648	677	686	715	745	774	803	83
28	643	671	686	708	736	764	793	82
29	632	660	668	696	723	751	779	80
30	621	649	657	684	711	738	766	79
31	610	637	645	[illegible]	698	725	752	77
32	599	626	634	[illegible]	685	712	738	76
33	589	615	622	648	673	698	725	75
34	578	603	610	636	660	685	711	73
35	567	592	599	624	648	672	698	72
Area, sq in	49.83	52.08	52.77	55.02	57.27	59.52	61.77	64.
I_{1-1}, in⁴.....	2 470	2 627	2 525	2 682	2 841	3 002	3 166	3 3
r_{1-1}, in	7.04	7.10	6.92	6.98	7.04	7.10	7.16	7.
I_{2-2}, in⁴.....	1 514	1 575	1 589	1 649	1 710	1 771	1 832	1 8
r_{2-2}, in.....	5.51	5.50	5.49	5.48	5.46	5.45	5.45	5.
Weight, lb per lin ft..	169.5	177.1	179.5	187.1	194.8	202.4	210.1	217

Safe load-values above the heavy line are for ratios of l/r not over 60; the below the heavy line are for ratios not over 120 l/r .

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVII* (Continued). Safe Loads in Units of 1 000 Pounds for 15-Inch Channel-Columns



Allowable fiber-stress in pounds per square inch:
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100\, l/r$$

Weights do not include rivet-heads or other details

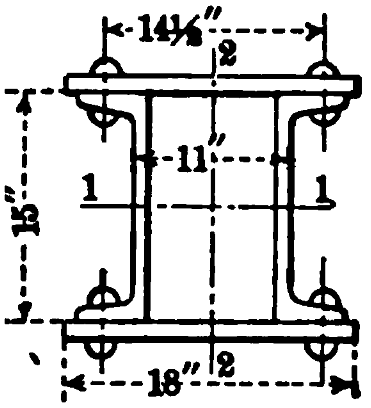
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 15-in channels, two 18-in plates						
	45-lb channels, 1 1/16-in plates	45-lb channels, 1 1/8-in plates	45-lb channels, 1 3/16-in plates	45-lb channels, 1 1/4-in plates	45-lb channels, 1 5/16-in plates	45-lb channels, 1 3/8-in plates	45-lb channels, 1 7/16-in plates
11	841	871	900	929	958	988	1 017
12	841	871	900	929	958	988	1 017
13	841	871	900	929	958	988	1 017
14	841	871	900	929	958	988	1 017
15	841	871	900	929	958	988	1 017
16	841	871	900	929	958	988	1 017
17	841	871	900	929	958	988	1 017
18	841	871	900	929	958	988	1 017
19	841	871	900	929	958	988	1 017
20	841	871	900	929	958	988	1 017
21	841	871	900	929	958	988	1 017
22	841	871	900	929	958	988	1 017
23	841	871	900	929	958	988	1 017
24	841	871	900	929	958	988	1 017
25	841	871	900	929	958	988	1 017
26	841	871	900	929	958	988	1 017
27	841	871	900	929	958	987	1 015
28	829	857	885	913	942	970	998
29	814	843	870	897	926	953	980
30	800	828	855	882	909	936	963
31	786	813	839	866	893	919	945
32	771	798	824	850	877	902	928
33	757	783	809	834	860	885	911
34	743	768	793	818	844	868	893
35	728	754	778	802	827	852	876
Area, sq in	64.73	66.98	69.23	71.48	73.73	75.98	78.23
I_{1-1}, in⁴	3 221	3 387	3 556	3 727	3 900	4 076	4 255
r_{1-1}, in	7.05	7.11	7.17	7.22	7.27	7.32	7.37
I_{2-2}, in⁴	1 903	1 964	2 025	2 086	2 146	2 207	2 268
r_{2-2}, in	5.42	5.42	5.41	5.40	5.40	5.39	5.38
Weight, lb per lin ft	220.1	227.7	235.4	243.0	250.0	258.3	266.0

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVII* (Continued). Safe Loads in Units of 1 000 Pounds for 15-in Channel-Columns



Allowable fiber-stress in pounds per square in
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13),

$$S = 19\,000 - 100\, l/r$$

Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 15-in channels, two 18-in plates						
	45-lb channels, 1½-in plates	45-lb channels, 1 9/16-in plates	45-lb channels, 1⅝-in plates	45-lb channels, 1 11/16-in plates	45-lb channels, 1¾-in plates	45-lb channels, 1⅞-in plates	45-lb channels, ...
11	1 046	1 075	1 105	1 134	1 163	1 222	1 28
12	1 046	1 075	1 105	1 134	1 163	1 222	1 28
13	1 046	1 075	1 105	1 134	1 163	1 222	1 28
14	1 046	1 075	1 105	1 134	1 163	1 222	1 28
15	1 046	1 075	1 105	1 134	1 163	1 222	1 28
16	1 046	1 075	1 105	1 134	1 163	1 222	1 28
17	1 046	1 075	1 105	1 134	1 163	1 222	1 28
18	1 046	1 075	1 105	1 134	1 163	1 222	1 28
19	1 046	1 075	1 105	1 134	1 163	1 222	1 28
20	1 046	1 075	1 105	1 134	1 163	1 222	1 28
21	1 046	1 075	1 105	1 134	1 163	1 222	1 28
22	1 046	1 075	1 105	1 134	1 163	1 222	1 28
23	1 046	1 075	1 105	1 134	1 163	1 222	1 28
24	1 046	1 075	1 105	1 134	1 163	1 222	1 28
25	1 046	1 075	1 105	1 134	1 163	1 222	1 28
26	1 046	1 075	1 105	1 134	1 163	1 222	1 28
27	1 044	1 073	1 102	1 131	1 159	1 216	1 27
28	1 026	1 054	1 083	1 112	1 139	1 195	1 25
29	1 009	1 036	1 064	1 092	1 119	1 174	1 23
30	991	1 017	1 045	1 073	1 099	1 153	1 20
31	973	999	1 026	1 053	1 079	1 132	1 18
32	955	980	1 007	1 034	1 059	1 111	1 16
33	937	962	988	1 014	1 039	1 090	1 14
34	919	943	969	995	1 019	1 069	1 12
35	901	925	950	975	999	1 048	1 09
Area, sq in	80.48	82.73	84.98	87.23	89.48	93.98	98.
I_{1-1}, in⁴.....	4 436	4 619	4 805	4 994	5 185	5 575	5 9
r_{1-1}, in.....	7.42	7.47	7.52	7.57	7.61	7.70	7.
I_{2-2}, in⁴.....	2 329	2 389	2 450	2 511	2 572	2 693	2 8
r_{2-2}, in.....	5.38	5.37	5.37	5.37	5.36	5.35	5.
Weight, lb per lin ft..	273.6	281.3	288.9	296.6	304.2	319.5	334

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

…nits of 1 000 Pounds for 15-Inch
…ns

…s in pounds per square inch:
…o radii or under
between 60 and 120 radii, by

9 000 — 100 l/r
rivet-heads or other details
…e 120, see notes on page 490

Effective length, ft	Two 15-in channels, 35 lb: 2 flange-plates 18"×2", 2 web-plates 14"×3/4"	Two 15-in channels, 35 lb: 2 flange-plates 18"×2", 2 web-plates 14"×15/16"	Two 15-in channels, 45 lb: 2 flange-plates 18"×2", 2 web-plates 14"×15/16"	Two 15-in, 45-lb channels: 2 flange-plates 20"×1 3/4", 2 web-plates 14"×5/8"	Two 15-in, 45-lb channels: 2 flange-plates 20"×2", 2 web-plates 14"×5/8"	Two 15-in, 45-lb channels: 2 flange-plates 20"×2 1/4", 2 web-plates 14"×5/8"	Two 15-in, 45-lb channels: 2 flange-plates 20"×2 1/2", 2 web-plates 14"×5/8"
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 340	1 408	1 485	1 547	1 612	1 677	1 742
	1 331	1 394	1 465	1 547	1 612	1 677	1 742
	1 307	1 369	1 439	1 547	1 612	1 677	1 742
	1 284	1 344	1 413	1 547	1 612	1 677	1 742
	1 261	1 320	1 387	1 543	1 607	1 670	1 735
	1 238	1 295	1 361	1 519	1 582	1 644	1 708
	1 214	1 270	1 335	1 495	1 557	1 618	1 681
33	1 191	1 246	1 309	1 471	1 532	1 592	1 654
34	1 168	1 221	1 283	1 447	1 507	1 566	1 627
35	1 145	1 197	1 257	1 424	1 482	1 540	1 600
…ea, sq in	103.08	108.33	114.23	118.98	123.98	128.98	133.98
	6 037	6 123	6 233	6 397	6 843	7 300	7 769
	7.65	7.52	7.39	7.33	7.43	7.52	7.61
	2 919	3 021	3 148	4 240	4 407	4 573	4 740
	5.32	5.28	5.25	5.97	5.96	5.95	5.95
	350.5	368.4	388.4	404.5	421.5	438.5	455.5

…-values above the heavy line are for ratios of l/r not over 60; those …heavy line are for ratios not over 120 l/r

…om Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XXVII* (Concluded). Safe Loads in Units of 1 000 Pounds for 15-in Channel-Columns

Allowable fiber-stress in pounds per square inch
13 000 for lengths of 60 radii or under
Reduced for lengths between 60 and 120 radii, by Formula (13).

$$S = 19\,000 - 100l/r$$

Weights do not include rivet-heads or other details
For values for l/r above 120, see notes on page 490

Effective length, ft	Two 15-in, 45-lb channels					
	2 flange-plates 20"×[illegible] 2 web-plates 14"×5/8"	2 flange-plates 20"×[illegible] 2 web-plates 14"×5/8"	2 flange-plates 20"×[illegible] 2 web-plates 14"×5/8"	2 flange-plates 20"×[illegible] 2 web-plates 14"×5/8"	2 flange-plates 20"×[illegible] 2 web-plates 14"×5/8"	2 flange-plates [illegible] 2 web-plates [illegible]
11	1 807	1 872	1 937	2 002	2 067	2 13
12	1 807	1 872	1 937	2 002	2 067	2 13
13	1 807	1 872	1 937	2 002	2 067	2 13
14	1 807	1 872	1 937	2 002	2 067	2 13
15	1 807	1 872	1 937	2 002	2 067	2 13
16	1 807	1 872	1 937	2 002	2 067	2 13
17	1 807	1 872	1 937	2 002	2 067	2 13
18	1 807	1 872	1 937	2 002	2 067	2 13
19	1 807	1 872	1 937	2 002	2 067	2 13
20	1 807	1 872	1 937	2 002	2 067	2 13
21	1 807	1 872	1 937	2 002	2 067	2 13
22	1 807	1 872	1 937	2 002	2 067	2 13
23	1 807	1 872	1 937	2 002	2 067	2 13
24	1 807	1 872	1 937	2 002	2 067	2 13
25	1 807	1 872	1 937	2 002	2 067	2 13
26	1 807	1 872	1 937	2 002	2 067	2 13
27	1 807	1 872	1 937	2 002	2 067	2 13
28	1 807	1 872	1 937	2 002	2 067	2 13
29	1 807	1 872	1 937	2 002	2 067	2 13
30	1 798	1 863	1 926	1 991	2 054	2 116
31	1 770	1 834	1 896	1 960	2 022	2 085
32	1 742	1 805	1 866	1 929	1 989	2 052
33	1 714	1 776	1 836	1 897	1 957	2 019
34	1 686	1 747	1 806	1 866	1 925	1 985
35	1 658	1 718	1 775	1 835	1 893	1 952
Area, sq in	138.98	143.98	148.98	153.98	158.98	163.9
I_{1-1}, in⁴	8 251	8 744	9 251	9 770	10 301	10 8
r_{1-1}, in	7.70	7.79	7.88	7.97	8.05	8.1
I_{2-2}, in⁴	4 907	5 073	5 240	5 407	5 573	5 7
r_{2-2}, in	5.94	5.94	5.93	5.93	5.92	5 9
Weight, lb per lin ft	472.5	489.5	506.5	523.5	540.5	557.

Safe load-values above the heavy line are for ratios of l/r not over 60; those below the heavy line are for ratios not over 120 l/r

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

CHAPTER XV

STRENGTH OF BEAMS AND BEAM GIRDERS. FRAMING AND CONNECTING STEEL BEAMS

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1. General Principles of the Flexure of Beams

Definitions. A structural member placed in a generally horizontal position upon two or more supports or projecting from some other construction is called a BEAM. A GIRDER is a beam carrying smaller or secondary beams. A CANTILEVER BEAM is a beam supported at the middle, or having one end fixed, as in a wall, and the other end free; or it is the part of a beam which overhangs, or projects, beyond a support. A SIMPLE BEAM is one which rests upon two supports, one at each end. A CONTINUOUS BEAM rests upon more than two supports. The distance between the supports of a simple beam, or, when so specially designated from center to center of the bearings, is the SPAN. It is usually designated by l. The loads on beams are either UNIFORMLY DISTRIBUTED or CONCENTRATED. A uniformly distributed or UNIFORM load includes the weight of the beam itself and any load spread evenly over it, such as the weight of a wall. Uniform loads are estimated by their intensity per unit of length of the beam, in pounds per linear foot. A uniform load per linear foot is represented by w, and the total uniform load by wl or W. A concentrated load is a single applied weight, such as a column and its load, or the load from another beam, and is designated by P.

Stresses and Deformations. A load on a simple beam causes the fibers to bend or deflect, and eventually to break across, or in other words, a load induces TRANSVERSE OR FLEXURAL STRESSES in the fibers. Since it is impossible to bend or deflect a simple beam without causing a shortening of the fibers on the upper or concave side and an elongation of the fibers on the lower or convex side, a load on a beam causes COMPRESSION in the upper fibers and TENSION in the lower fibers, while between the two there is a neutral layer or surface of fibers which is unchanged in length and which is called the NEUTRAL SURFACE of the beam. In a cantilever beam the reverse is the case, the upper fibers being in tension and the lower ones in compression.

Laws Determined by Experiment. From experiments it has been found that the amount of elongation or shortening of any fiber is directly proportional to its distance from the neutral surface of a beam; hence, if the ELASTIC LIMIT is not exceeded, the stresses, also, are proportional to their distances from the neutral surface. The trace of the neutral surface on a cross-section of a beam is called the NEUTRAL AXIS of the cross-section. Within the elastic limit of a material the neutral surface passes through the CENTERS OF GRAVITY of the cross-sections of a beam for all materials.

Bending Moments and Resisting Moments.* To determine the strength of any beam to resist the effects of any load or series of loads, two things must

* See, also, Chapter IX, pages 324 to 331.

be determined: first, the moment or moments of the external destructive fo or forces tending to bend and break the beam, which is called the MAXIM BENDING MOMENT; and, secondly, the moments of the combined resistance all the fibers in the DANGEROUS SECTION of the beam to being broken, wh in their summation, are called the MOMENT OF RESISTANCE or the RESIST MOMENT.

The Methods of Finding the Bending Moments for any load or se of loads are explained in Chapter IX. The moment of resistance is equa the SECTION-MODULUS or SECTION-FACTOR, denoted by I/c, multiplied by unit stress on the outermost fiber of the material, denoted by S, and it eq the bending moment.

Hence $$M = SI/c$$

This is known as the FLEXURE-FORMULA and it is the fundamental formula designing beams. Formulas for finding the section-moduli of common sh are given in Chapter X, and the values of I/c or the section-moduli of standard rolled shapes, are given in the tables in the same chapter.

The Coefficient of Strength,* sometimes given in tables of steel beam the maximum distributed load that a beam of one foot span would sup without producing a fiber-stress exceeding the safe limit, generally 16 oo per sq in. As the strength of a beam varies inversely as its span, the safe for any span may be obtained by dividing this coefficient by the span in fee

Factors of Safety. In order that a beam shall just be able to carry a and not break, that condition of equilibrium must exist, in which the m mum bending moment in the beam is equal to the section-modulus multi by the ultimate strength of the material. In order that a beam may abundantly SAFE to carry a given load, the product of the section-modulu the ultimate strength of the material must be several times greater than maximum bending moment; and the ratio which this product bears to maximum bending moment, or which the BREAKING-LOAD bears to the s LOAD, is known as the FACTOR OF SAFETY, that is,

$$\text{Factor of safety} = \frac{\text{ultimate strength}}{\text{working stress}}$$

Ultimate Strengths and Safe Fiber-Stresses. By the STRENGTH OF MATERIAL is meant a certain constant quantity which is determined by ex ment, and which is known as the ULTIMATE BREAKING STRENGTH. value is of course different for each material. Table I gives the values of constant divided by the factor of safety, or in other words, the WORKING ST for most of the materials used in building-construction. The section-m multiplied by these values will give the SAFE RESISTING MOMENTS for the be The values of S in Table I for steel are about one-fourth those of the break loads; for cast iron, about one-sixth; for average specimens of wood, one-s and for stone and concrete, one-tenth. The safe compressive strength of iron for the compression-side of beams is 16 000 lb per sq in, in the New Building Code. This is considered too high by some engineers and the au recommends 10 000 lb per sq in. This value has been used in calculating safe loads for cast-iron columns. (See Chapter XIV, page 461.) The loads for the steel shapes given in the tables in this chapter are all comp

* The values for coefficients of strength have been omitted from most of the t following the policy of some of the latest handbooks, as the safe loads for beam example, can be as readily determined from the data of the tables directly, as b process of dividing such coefficients by the spans. See, however, pages 586 to 59 623 to 628.

the value of 16 000 lb per sq in for S, but these full loads should be used h caution, and reduced when necessary to satisfy any unusual conditions. r riveted steel girders 14 000 lb per sq in was the value formerly given to S, t the usual value now is 16 000 lb per sq in.

Table I. Safe Unit Fiber-Stresses, S, for Flexure of Beams *

s to be noted that these are average values, especially those for wood. For allowable higher stresses for timber, see also, notes on pages 628, 637 and 647.

Materials Wood unseasoned †	Values of S, lb per sq in	Materials Wood unseasoned †	Values of S, lb per sq in
ast iron, tension-side	3 000	Redwood, California	750
ast iron, compression-side	16 000	Short-leaf yellow pine	1 000
rought iron (rolled beams)	12 000	Spruce	700
eel (rolled beams)	16 000	White oak	1 200
eel (riveted girders) both flanges	16 000	White pine	700
eel (pins, rivets and bolts)	24 000	Bluestone flagging (North River)	305
dar	700	Brick (common)	50
estnut	800	Brickwork (in cement)	30
press	800	Granite (average)	180
uglas fir	1 000	Limestone (average)	145
m	900	Marble (average)	125
mlock	600	Sandstone (average)	110
cust	1 200	Slate (average)	400
g-leaf yellow pine	1 200	Concrete (Portland) 1 : 2 : 4	30
rway pine	800	Concrete (Portland) 1 : 2 : 5	20
		Concrete (natural) 1:2:4	16
		Concrete (natural) 1:2:5	10

† For a comparison of values given in different building laws see Table XVII, page 648, pter XVI. Compare, also, with Table XVI, page 647, Chapter XVI. For ultimate es for woods, see Tables XVIII and XIX, pages 650 and 651, Chapter XVI. For loads for unit beams, see Tables II and III, page 628, Chapter XVI.

Add from 30 to 40% for seasoned, protected timber, used without impact.

ams Unsymmetrically Loaded or of Irregular Cross-Section. There ertain loadings and cross-sections of beams that occur most frequently in ing-construction, and for which tables have been worked out that give the loads directly; but for a beam unsymmetrically loaded, or for a beam of ular cross-section, it is impossible to compute tables for strength, as in each the values must be computed by determining either the section-modulus, quired to resist the maximum bending moment, or the maximum bend-oment that may be allowed for a given value of the section-modulus.

eral Formulas for the Flexure of Beams.* The general formula for eam in a state of flexure under any system of loading is

Maximum bending moment in INCH-POUNDS = section-modulus × S (2)

$$M_{\max} = SI/c \qquad (2)'$$

$$\text{Section-modulus} = \frac{\text{maximum bending moment in in-lb}}{S} \qquad (3)$$

$$I/c = M_{\max}/S \qquad (3)'$$

* See, also, Chapters IX, X and XVI.

If the bending moment is computed in FOOT-POUNDS, these formulas beco

$$\text{Maximum bending moment} = \frac{\text{section-modulus} \times S}{12}$$

or

$$M_{max} = SI/12\,c$$ (

and

$$\text{Section-modulus} = \frac{12 \times \text{maximum bending moment}}{S}$$

or

$$I/c = 12\,M_{max}/S$$ (

By substituting for the bending moments their values in terms of the lo and the spans, the following formulas which apply to beams of any cross-sect are readily deduced.

2. Formulas for Safe Loads for Beams for Different Conditions of Loading and Support

I/c = the section-modulus,
S = the safe unit fiber-stress in pounds per square inch;
W = the total uniform load in pounds;
P = the concentrated load in pounds;
l = the span in feet.

Values of I/c for the various shapes and sizes of structural-steel shapes given in the tables of Chapter X.

Case I

Beam Fixed at One End and Loaded with a Concentrated Load P, Near the I End (Fig. 1).

From Formula (4)′,

$$M_{max} = SI/12\,c$$

From Case I, Chapter IX,

$$M_{max} = Pl$$

Hence

$$Pl = SI/12\,c$$

and the safe load in pounds is

$$P = SI/12\,cl$$

and the section-modulus is

$$I/c = 12\,Pl/S$$

Fig. 1. Cantilever Beam. Load near Free End

Example 1. A steel T bar is fixed at one end in a brick wall, and loade the other end with 600 lb, the distance l being 4 ft. What is the size of the required to support the load with safety? (In all examples the weights of beams are neglected, unless particularly mentioned.)

Solution. Allowing 16 000 lb per sq in for the value of S, Formula (6)′ ,

$$I/c = (12 \times 600 \times 4)/16\,000 = 1.8$$

The next step is to ascertain what T bar has a section-modulus equal tc In Table XIV, page 369, the nearest section-modulus to this is 1.9, corresp ing to a 3 by 4 by ½-in T bar.

For an I beam, by Table IV, page 355, $I/c = 1.8$, the same as for the T and calls for a 3-in 6.5-lb I beam.

Case II

...m Fixed at One End and Loaded with a Uniformly Distributed Load W (...g. 2).

...rom Formula (4)′

$$M_{max} = SI/12c$$

...m Case II, Chapter IX,

$$M_{max} = Wl/2$$

...nce

$$Wl/2 = SI/12c$$

... the safe load in pounds is

$$W = SI/6cl \qquad (7)$$

...d

$$I/c = 6Wl/S \qquad (7)'$$

Fig. 2. Cantilever Beam. Distributed Load over Entire Span

...ample 2. What is the size of a can...ver steel I beam required to carry a uniformly distributed load of 150 lb per ...over a length of 6 ft?

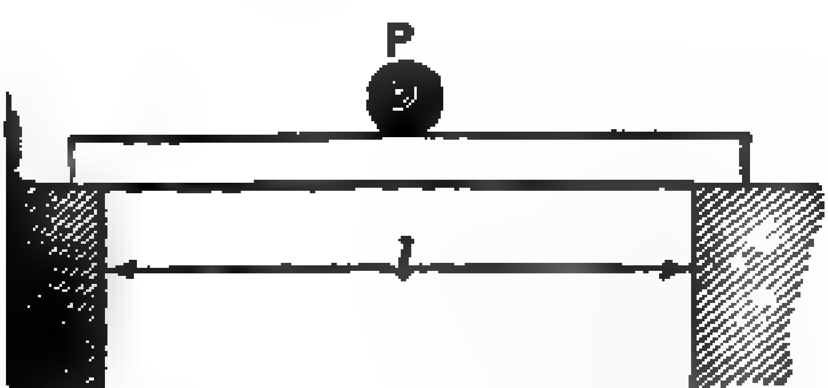
...lution. $W = 150 \times 6 = 900$ lb. Substituting in Formula (7)′,

$$I/c = \frac{6 \times 900 \times 6}{16\,000} = 2.025$$

... Table IV, page 355, the nearest section-modulus to this is 1.9, which is ...t of a 3-in 7.5-lb beam, the heaviest of that depth. However, as the ...est 4-in beam, also, weighs 7.5 lb per ft it probably would be selected be...se of its greater stiffness, although its section-modulus is 3, still greater ...n required.

Case III

...m Supported at Both Ends and Loaded with a Concentrated Load at the ...ddle (Fig. 3).



...g. 3. Simple Beam. Load at Middle of Span

From Formula (4)′

$$M_{max} = SI/12c$$

From Case IV, Chapter IX,

$$M_{max} = Pl/4$$

Hence

$$Pl/4 = SI/12c$$

and the safe load in pounds is

$$P = SI/3cl \qquad (8)$$

and

$$I/c = 3Pl/S \qquad (8)'$$

...ample 3. What steel I beam will safely support a concentrated load of ...ns applied at the middle of a 15-ft span?

...lution. $P = 7$ tons $= 14\,000$ lb. Substituting in Formula (8)′,

$$I/c = \frac{3 \times 14\,000 \times 15}{16\,000} = 39.3$$

...ing again to Table IV, page 355, it is seen that a 12-in 35-lb beam has

a section-modulus of 37.8, while a 12-in 40.8-lb beam, the next larger has a section-modulus of 44.8. The 35-lb beam, however, would undoubt be safe.

Case IV

Beam Supported at Both Ends and Loaded with a Uniformly Distributed (Fig. 4).

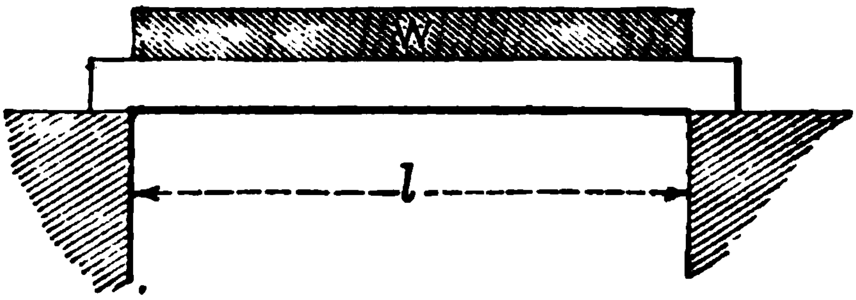


Fig. 4. Simple Beam. Distributed Load over Entire Span

From Formula (4)'

$$M_{max} = SI/12c$$

From Case V, Chapter IX,

$$M_{max} = Wl/8$$

Hence

$$Wl/8 = SI/12c$$

and the safe load in pounds is

$$W = 2SI/3d$$

and

$$I/c = 3Wl/2S$$

Example 4. What steel I beam will safely carry a uniformly distribute of 1 000 lb per ft over a span of 25 ft?

Solution. $W = wl = 1\,000 \times 25 = 25\,000$ lb. Substituting in Formula

$$I/c = \frac{3 \times 25\,000 \times 25}{2 \times 16\,000} = 58.6$$

From Table IV, page 354, the nearest section-modulus is 58.9, which is t a 15-in 42.9-lb beam.

Case V

Beam Supported at Both Ends and Loaded with a Distributed Load Over of the Span (Fig. 5).

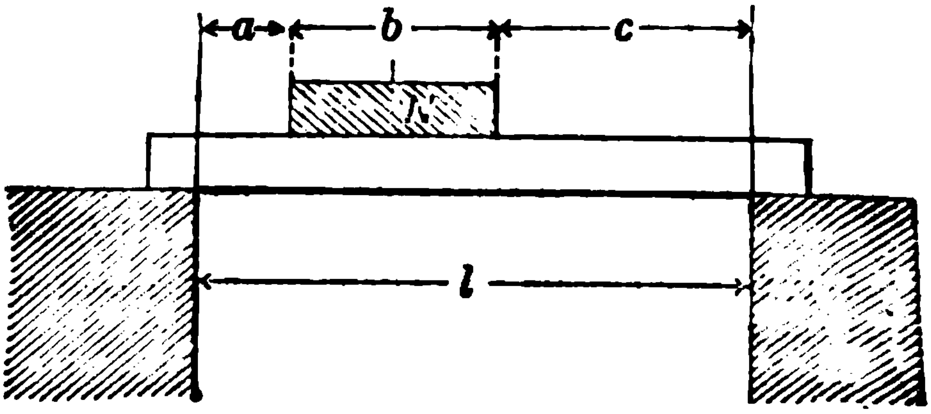


Fig. 5. Simple Beam. Distributed Load over Part of Span

In this case the load is generally given, and the problem is to determ size of the required beam. This can be done accurately only by com

maximum bending moment as explained for Case VIII, Chapter IX, and substituting the value thus found in Formulas (3)′ or (5)′.

Example 5. What steel I beam will safely carry a uniformly distributed load of 1 200 lb per ft over part of the span, beginning at a point 5 ft from the left reaction and extending over a distance of 6 ft, the span of the beam being 18 ft?

Solution. The first step is to find the point of maximum bending moment, which is the point of no shear. Obviously the maximum shear is just at the right of the reaction nearest the load, which in this case is the left reaction. To find the left reaction (see Chapter IX, page 324) the center of moments is taken at the right reaction and the equation of moments is $R_1 \times 18$ ft $-$ (1 200 lb $\times$ 6 ft) $\times$ 10 ft $= 0$. $18\,R_1 = 72\,000$ and $R_1 = 4\,000$ lb. The shear just at the right of R_1 is therefore $+4\,000$ lb which, if the weight of the beam itself is not considered, remains unchanged for every section of the beam between the left reaction and the uniformly distributed load of 1 200 lb per ft. From there on in passing to the right, the shear is diminished at the rate of 1 200 lb per ft; and it becomes zero, therefore, at a point 4 000 lb/1 200 lb per ft = 3.3 ft to the right of the 5-ft point. Hence the point of no shear and consequently the point of maximum bending moment is at 5 ft + 3.3 ft, or 8.3 ft, from the left end. The equation for the maximum bending moment at this point is, therefore,

$$M_{max} = 4\,000 \text{ lb} \times 8.3 \text{ ft} - (1\,200 \text{ lb} \times 3.3 \text{ ft}) \times 3.3/2 \text{ ft}$$
$$= 33\,200 \text{ ft-lb} - 6\,534 \text{ ft-lb} = 26\,666 \text{ ft-lb, or } 319\,992 \text{ in-lb}$$

From Formula (3), $I/c = 319\,992$ in-lb/16 000 lb per sq in $= 20$. From Table V, page 355, the nearest section-modulus corresponding to this is 20.3, that of a 9-in 25-lb beam. A 10-in 25.4-lb beam, however, being stronger and stiffer, would probably be used. The 10-in 22.24-lb beam is what is termed a SUPPLEMENTARY BEAM. (See Case VIII, Chapter IX, and pages 352 and 353.)

Case VI

Beam Supported at Both Ends and Loaded with a Concentrated Load, not at the Middle (Fig. 6).

Fig. 6. Simple Beam. Concentrated Load at any Point

From Formula (4)′,

$$M_{max} = SI/12c$$

From Case VI, Chapter IX,

$$M_{max} = Pmn/l$$

Hence

$$Pmn/l = SI/12c$$

The safe load in pounds is

$$P = SIl/12\,cmn \qquad (10)$$

$$I/c = 12\,Pmn/lS \qquad (10)'$$

m, n and l being in feet.

Example 6. A steel I beam 20 ft in span is to support a concentrated lo 24 000 lb at a distance of 6 ft from the left support. What must be th and weight of the beam?

Solution. In this case $P = 24\,000$ lb, $l = 20$ ft, $m = 6$ ft, $n = 14$ ft and 16 000 lb per sq in.

Then Formula (10)′ gives

$$I/c = \frac{12 \times 24\,000 \times 6 \times 14}{20 \times 16\,000} = 75.6$$

Table IV, page 354, the nearest value for the section-modulus I/c fo 1–1 is above 75.6, or 81.2 for a 15-in 60.8-lb beam. An 18-in 55-lb beam h a section-modulus of 88.4 would be used, unless conditions fix the head- as it weighs 5 lb per ft less, and being deeper is consequently stiffer.

Case VII

Beam Supported at Both Ends and Loaded Symmetrically with Two Concentrated Loads (Fig. 7).

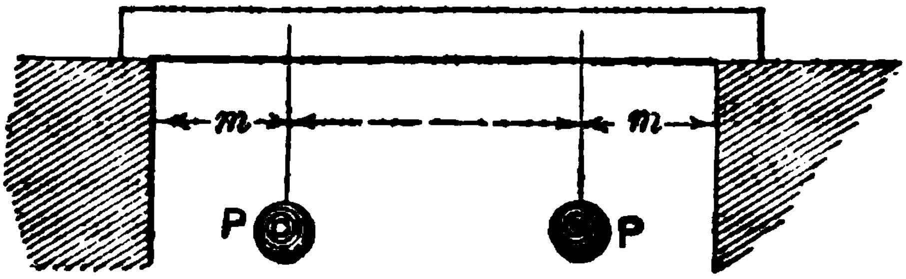


Fig. 7. Simple Beam. Equal Concentrated Loads Symmetrically Placed

From Formula (4)′ and Case VII, Chapter IX, each of the safe loads in p is

$$P = SI/12\,cm$$

and

$$I/c = 12\,Pm/S$$

Example 7. A 12-in steel channel, 12 ft in span, supports half the lo two 10-in beams 4 ft from each end. Each beam is designed to carry 16 o What is the size and the weight of the channel required?

Solution. The channel supports only one-half the load on each beam; $P = 8\,000$ lb, $m = 4$ ft, $S = 16\,000$ lb per sq in, and by Formula (11)′,

$$I/c = \frac{12 \times 8\,000 \times 4}{16\,000} = 24,$$

which is the section-modulus of a 12-in 25-lb channel. (See Table page 359.) Exact table-value, 23.9.

Weights of Beams in Flexure-Formulas. It will be noticed that i mulas (11) and (11)′ the span of the beam is not taken into account, and beam itself had no weight there would be no difference in the fiber-st no matter how far apart the loads P were placed. In reality, however, beams have considerable weight, and to be absolutely correct an example as the one above should include the weight of the beam, which would, of c be a uniformly distributed load. The maximum bending moment of the can be found graphically as explained on page 329, and the value of I/c puted by Formulas (3)′ or (5)′. Where, however, the loads are spaced to divide the beam into three equal parts, as in the last example, one-th the weight of the beam may be added to P with sufficient accuracy. Thu

ght of the channel in the above example between the supports would be lb × 12, or 300 lb, and P would be 8 100 lb, which would give a value for of 24.1. The factor of safety in the loads allowed is generally large enough offset the slight effect produced by the weight of the beam; but if the full d assumed is likely to be imposed on the beam, then allowance must be made the weight of the beam itself.

Case VIII

am Supported at Both Ends and Loaded Symmetrically with Several Con- ntrated Loads (Fig. 8).

In this case it is necessary to compute the maximum bending moment in the am and proportion the beam by Formulas (3)′ or (5)′.

Example 8. A steel-beam girder is to be designed to support a brick wall, in thick and weighing 138 000 lb, over an opening 22 ft wide. The girder t also support the ends four 10-in floor-beams ced as in Fig. 8, each m carrying 16 000 lb. hat is the size and weight the girder required?

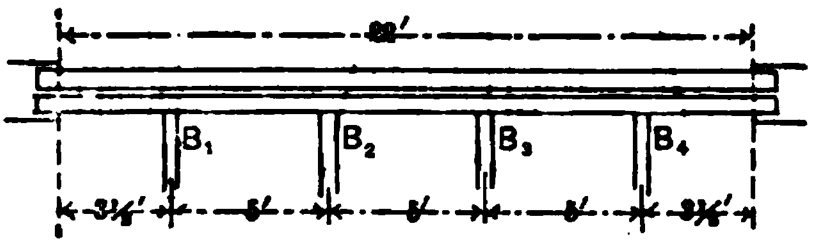


Fig. 8. Simple Beam. Several Concentrated Loads Symmetrically Placed

Solution. The first step to make an allowance for e weight of the girder. e total load on the girder (neglecting the weight of the girder itself) = 138 000 + 4 × 8 000 lb (one-half the load on each beam) = 170 000 lb, or 85 tons. this is much more than the heaviest single rolled beam will carry, it will be ssary to use a pair of beams and the load on each beam, therefore, will be tons. Considering for the present the entire load as uniformly distributed, le IV, page 577, shows that to support 42.5 tons, or 85 000 lb, over a span 2 ft requires a 24-in 85-lb beam. The girder then will weigh between sup- s 2 × 85 × 22 = 3 740 lb, or about 4 000 lb. This added to the weight of wall makes, for the total distributed load, 142 000 lb. The next step is to ermine the maximum bending moment.

By the formulas given in Chapter IX the maximum bending moments for various loads may be found as follows:

the wall and girder (Case V, page 326),

$$M_{max} = \frac{22 \times 142\,000}{8} = 390\,500 \text{ ft-lb}$$

the beam B_1 (Case VI, page 327),

$$M_{1\,max} = \frac{8\,000 \times 3\tfrac{1}{2} \times 18\tfrac{1}{2}}{22} = 23\,545 \text{ ft-lb}$$

the beam B_2 (Case VI, page 327),

$$M_{2\,max} = \frac{8\,000 \times 8\tfrac{1}{2} \times 13\tfrac{1}{2}}{22} = 41\,727 \text{ ft-lb}$$

e beams being spaced symmetrically from the middle of the span, the bend- oments for B_3 and B_4 will be equal to those of B_2 and B_1 respectively. ing the bending moments to a scale, in the manner explained for Figs. 17 page 330, the diagram shown in Fig. 9 is obtained. The greatest bend- oment is the ordinate M_x, which scales 486 500 ft-lb, or 5 838 000 in-lb.

Note. Since the loads are symmetrically placed, this ordinate is over middle point of the girder, but it is drawn to one side in the figure in order to confuse it with the ordinate M, the maximum bending moment for the formly distributed load. Substituting this value of M_x in formula (3)′,

$$I/c = \frac{5\ 838\ 000 \text{ in-lb}}{16\ 000 \text{ lb per sq in}} = 365$$

the section-modulus for both beams, or 182.5 for one beam. From Table page 354, it is found that a 24-in 90-lb beam has a section-modulus of 18 and two 90-lb beams will just ans The assumption of a uniform distribu of such a loading over every foot girder usually results in the selectio lighter beams than are indicated by second solution, in which each con trated load is considered as really centrated at a point. The two be should be securely bolted together separators near each connection of be B_1, B_2, B_3, B_4, and at each end of girder.

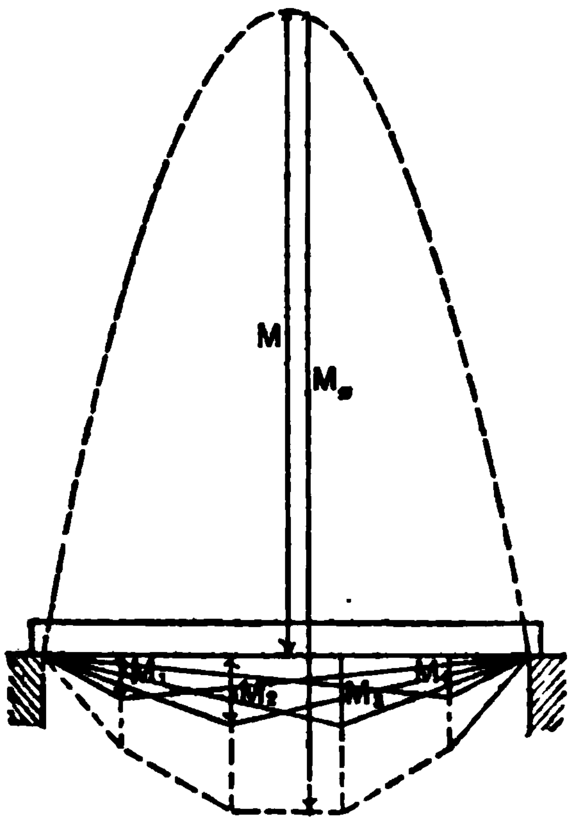


Fig. 9. Bending-moment Diagram for Beam Shown in Fig. 8.

A DOUBLE-BEAM GIRDER, however not considered the best kind of girde use under this condition of loading, as not good construction nor economica material. As a general rule BEAM DERS should be used only when the l can be applied to the upper flange both beams. Transferring a load dire to the web of one beam, even thoug is connected with the other beam means of separators, does not insur equal distribution of the loading. The author, therefore, recommends in case a RIVETED BEAM GIRDER or a RIVETED PLATE GIRDER. The method al indicated applies to any method of loading, the only difference in the culation being in the determination of the maximum bending moments.

Inclined Beams. The strength of beams inclined to the horizontal ma computed, with sufficient accuracy for most purposes, by using the form given for horizontal beams, and taking the HORIZONTAL PROJECTIONS of the b as the spans.

2. Steel Beams and Girders*

Materials Used for Beams. Practically the only materials used in s tural work for beams, at the present day, are wood, steel and reinforced conc As wooden beams are always rectangular in cross-section, the general form used in this chapter can be much simplified by substituting for I/c its val terms of the breadth and depth of the beam. Formulas for wooden beam therefore be found in Chapter XVI. Cast iron, also, is occasionally use beams or lintels, but as this material is much stronger in resisting compre than tension, the beam must be of a special shape in order to use the ma to advantage. The strength of cast-iron beams is therefore considered

* For the deflection of steel beams, see Chapter XVIII.

pecial heading in Chapter XVI. Formulas for reinforced-concrete beams are ven in Chapter XXIV, pages 924 to 939; and Chapter XXV, page 992.

Forms of Steel Beams. Since 1893, steel beams have superseded wrought-on beams, and the latter are now never used. Any shape of rolled steel may e used as a beam, but the I shape is the most economical, as it possesses the eatest resistance for a given weight of metal. Next to the I beam, in economy, the channel, then the deck beam; angles and tees are the least economical all shapes. The following values show the safe loads per pound of steel, for e various shapes, for a 10-ft span; the same ratio would hold for other spans.

10-in I beam	10-in channel	10-in deck-beam	4 by 6-in angle	4 by 5-in tee
104	94.6	83.0	28.7	21.6

The Deepest Beams, the Strongest, Stiffest and Most Economical. e STRENGTH of a wooden or steel beam of rectangular cross-section varies as e SQUARE OF THE DEPTH, directly as the breadth and inversely as the length, d the STIFFNESS varies directly as the CUBE OF THE DEPTH, directly as the adth and inversely as the cube of its length; hence the deeper beam will e the greater strength and stiffness in proportion to its sectional area. With eams these relations do not hold strictly, because of the variation in the ms of the cross-sections, but they are approximately true. It therefore ows that, for any given span, it is more economical in floors, where other ditions will permit, to use deep beams spaced farther apart or to use one deep m in place of two shallower beams. Thus if a distributed load of 39 tons o be supported over a span of 16 ft, one 20-in 65-lb beam, two 15-in 42-lb ms, or three 12-in 40-lb beams, could be used; but the 20-in beam would h only 1 105 lb, allowing for 6-in bearings, as compared with 1 428 lb for 15-in beams and 2 040 lb for the 12-in beams, and the bolts and separators ld be saved.

ight and Heavy Steel Beams. LIGHT BEAMS are more economical than vy beams OF THE SAME DEPTH, except when the span is so short that the load is governed by the resistance of the web to buckling, in which case the VY BEAMS are the more economical.

aximum Safe Loads for Steel Beams. All loaded beams are, in general, ject to three kinds of stresses. The most destructive are generally those e to the BENDING MOMENTS, and have already been considered. The second s are those which tend to SHEAR a beam, or to make one part slide on other vertically. (See paragraph on Shearing-Stresses in Steel Beams and ders, page 567.) These stresses, however, seldom need to be considered pt in the case of riveted girders and short beams with very thick webs. e third kind of stress is that which tends to cause the web of a beam to KLE; and in a steel beam over a span very short in proportion to the th of the beam, the resistance of the web to buckling generally determines maximum load that the beam, without stiffeners on the web, will support. e also, pages 182, 183 and 567.)

e Loads for Steel Beams.* To save time in calculating, tables of safe s for structural and supplementary beams and channels used as beams r conditions of transverse loading, have been prepared, which give the UNI-LY DISTRIBUTED SAFE LOADS in thousands of pounds for spans customary

* Part of the matter of the following paragraphs relating to steel I beams has been ed by permission, from the Pocket Companion, Carnegie Steel Company, Pitts-, Pa.

in building-construction. They are based upon an extreme FIBER-STRESS 16 000 lb per sq in on the fibers farthest from the neutral surface of the be

The Tables of Safe Loads for Angles and Tees, pages 586 to 591, the values at the same fiber-stress on spans of one foot, from which the safe for any span-length may be obtained by direct division, and also the values those spans at which the allowed safe load will produce a deflection of ⅟₃₆₀ the span-length. The loads in all cases include the weight of the beam, w should be deducted in order to arrive at the net load which the beam will supp For several concentrated loads or for a combination of distributed and con trated loads it will be necessary to use the methods previously explained u Case VIII, page 563.

Use of Tables for Concentrated Loads. To use any of the follo tables for CONCENTRATED LOADS, find the equivalent distributed load by m plying the concentrated load by the factor given in Table IV, page 632, then use the beam having a safe load equal to the load thus found.

In addition to the conversion-factors in that table the following, also, wi found convenient:

For two equal loads applied at one-third the span from each end, multiply load by 2⅔.

For two equal loads applied at one-fourth the span from each end mul one load by 2.

For a beam fixed at one end, and loaded at the other, multiply by 8.

For a beam fixed at one end, and uniformly loaded over the entire le multiply by 4.

Unusual Conditions of Loading of Beams.* It is assumed in all that the loads are applied normal to the axis 1–1 as shown in the tables o properties of sections in Chapter X, and that the beam deflects vertically i PLANE OF BENDING ONLY. If the conditions of loading involve the intr tion of forces outside this plane of loading, the allowable safe loads mu determined from the general theory of flexure in accordance with the mo application of the load and its character. This applies particularly to un METRICAL SECTIONS, such as angles, which should be used under those c tions of loading where the section can deflect vertically only, being rigid cured against LATERAL DEFLECTION or twisting throughout the entire In all such cases of eccentric loading, the actual safe loads would be conside lower than the tabulated safe loads, which have been based upon the favorable conditions of loading.

Vertical Deflection of Steel Beams.* In the case of beams intend carry plastered ceilings, experience indicates that the VERTICAL DEFLEC to avoid cracking the plaster, should be limited to not more than ⅟₃₆₀ spain-length. This SPAN-LIMIT for steel beams is approximately, in feet, the depth in inches and is indicated in the tables by the lower, broken, zontal lines. Beams intended for such purposes should not be used for g spans unless the allowable tabular safe load exceeds the actual load to b ported. As the dead load of a floor is supported by the floor-beams befo plaster is applied, only the deflection due to the live load really needs considered. The vertical deflection of beams is explained in Chapter XV

Lateral Deflection of Steel Beams.* The tabular safe loads are upon the assumption that the compression-flanges of the various sectio

* Part of the matter of this paragraph has been adapted, by permission, from the Companion, Carnegie Steel Company, Pittsburgh, Pa.

...ured at proper intervals, against LATERAL DEFLECTION, by the use of tie-rods ... by other means. The LATERAL UNBRACED LENGTH of steel beams and ...ders should not exceed forty times the width of the compression-flanges. ...hen the unbraced length exceeds ten times the width, the tabular safe loads ...uld be reduced. An explanation of the method of reducing the tabular loads ...en the unsupported length exceeds ten times the flange-width is given in ...pter XVIII, page 670. (See Bethlehem Handbook for sidewise deflection.)

...Shearing-Stresses in Steel Beams and Girders.* The safe-load tables for ...ms and channels are computed solely with reference to SAFE UNIT STRESSES ...E TO FLEXURE, and the safe loads uniformly distributed on the spans given ...l not cause average SHEARING-STRESSES in the web greater than the 10 000 ...per sq in, the average SAFE WORKING STRENGTH of steel in SHEAR. When, ...ever, beams are loaded with heavy loads concentrated near the supports, ...when beams of short span are loaded with uniformly distributed loads to their ... carrying capacity as regards flexure, the bending moments may be small ...comparison with the reactions at the supports, and the beams may fail along ...neutral surface as a result of LONGITUDINAL SHEARING-STRESSES, or they may ...CKLE as a result of the combined longitudinal and vertical web-stresses. ... such spans the safe shearing or buckling strength of the web rather than ...resistance of the flanges to bending-stresses may limit the carrying capacity ...the beam.

...ckling Values of Beam-Webs.* The VERTICAL SHEARING-STRESSES or ... vertical compressive components of the web-stresses may under some con...ns exceed the safe resistance of the beam to BUCKLING, and there remains ... possibility that a web or web-plate, which is amply secure against the safe ...ed shear of 10 000 lb per sq in, will not be of sufficient strength when con...ed as a column. In such cases provision must be made for security against ...LING either by stiffeners or by an increased thickness of the web or web-... (For the determining conditions for web-buckling of steel beams in gril..., based on direct compression, see page 183.)

...ditions of Web-Buckling of Steel Beams. There are two conditions ...EB-BUCKLING (see, also, foot-note for paragraphs relating to Tables II and ...

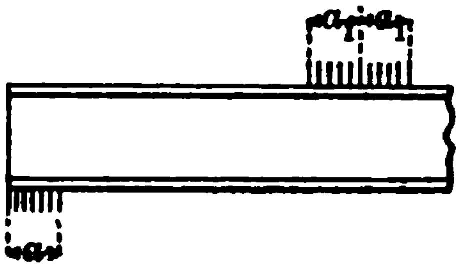
... The part of the beam bearing on the support is subject to DIRECT COM...SION, and the web over this part must be capable of resisting it. If this ...is too small the end of the beam will fail, as a column, causing the web to ...KLE. It is therefore necessary to calculate the required length of the bear-...

...) The beam throughout its length between the supports, or in case of a ...ever beam, from its end to the support, is subject to SHEAR. It is gen...y supposed that the shear develops stresses of TENSION and COMPRESSION ...e web; that these stresses act at right-angles to each other in the plane of ...eb and at an angle of 45° with the neutral surface of the beam; and that ... DIAGONAL STRESSES are equal in magnitude or intensity to the VERTICAL ... at any point. It is the COMPRESSIVE STRESS that tends to BUCKLE the ...

...ulas for Safe Buckling Resistance of Steel Beams.* In regard to ...st condition of buckling a series of experiments has been made on beams ...ious depths and web-thicknesses to arrive at a basis for a simpler method ...putation to use in the investigation of the safe BUCKLING RESISTANCE of ...

* ...t of the matter of this paragraph has been adapted, by permission, from the ... Companion, Carnegie Steel Company, Pittsburgh, Pa.

beams with unsupported webs, and from these experiments the following
mulas * have been deduced:



Safe end-reaction $R = S_b \times t\left(a + \frac{d}{4}\right)$

Safe interior load $P = 2\,S_b \times t\left(a_1 + \frac{d}{4}\right)$

In these formulas, R is the end-reaction, P the concentrated load, t the
thickness, d the depth of the beam, a_1 half the distance over which the con
trated load is applied and a the whole distance over which the end-reacti
applied; while S_b is the SAFE RESISTANCE OF THE WEB TO BUCKLING, in po
per square inch, by the straight-line formula

$$S_b = 19\,000 - 100\,d/2\,r$$

$d/2 = l$ in the column-formula †. The first formula is general and appli
any condition of loading. The second formula covers the case of a single
concentrated at the middle of a span; it can be extended to cover a syste
concentrated loads provided the sum of the distances a_1 is not less than a.

Tables II ‡ and III ‡ give for beams and channels with unsupp
webs:

* These formulas, in order to satisfy the first condition, are used in the Pocket
panion, 1915 Edition, Carnegie Steel Company, Pittsburgh, Pa.

† This is the column-formula used by the American Bridge Company and in Carn
Pocket Companion, S being the allowable COMPRESSIVE UNIT STRESS in pounds
square inch within the usual limits of l/r. See Formula (13), page 481.

‡ In regard to the shearing of steel beams, allowable web-shears, etc., the valu
example (see Example 15, this chapter, and on pages 182 and 183 of Chapter
42 000 lb per sq in for a 12-in, 31.8-lb I beam, given in Table II, page 575, takes
Carnegie's Pocket Companion, is based on the allowed direct shear without includi
condition of web-crippling. That is, the 42 000 lb is determined by taking the
the web, 0.35 × 12 = 4.2 sq in and multiplying it by 10 000 lb per sq in, which
value there used for the safe unit shearing-stress.

The beam is therefore calculated as being good for 42 000 lb SHEAR, but it is nec
to make a further investigation to ascertain whether the stresses due to shear will
the web of the beam to buckle. As stated in the paragraph on page 567, on the Bu
Values of Beam-Webs there are two conditions of web-buckling or web-crippling.

In the case of a plate girder the end-stiffeners provide for the first condition, a
intermediate stiffeners for the second condition. The web itself may then be coun
for its full shearing value. In the case of beams, however, it is not generally econ
to use stiffeners, so that the web alone must meet every condition.

The Carnegie Pocket Companion gives a formula, reproduced in the preceding
graph, and gives the derived lengths of bearings in Tables II and III, to satisfy t
condition. Some of the formulas used in the manufacturers' handbooks, for ma
safe shear based on web-buckling for the second condition, are as follows:

Passaic Steel Company, $V = \dfrac{10\,000\,dt}{1 + \dfrac{h^2}{3\,000\,t^2}}$

Cambria Steel Company, $V = \dfrac{12\,000\,dt}{1 + \dfrac{h^2}{1\,500\,t^2}}$

Bethlehem Steel Company, $V = \dfrac{12\,000\,dt}{1 + \dfrac{h^2}{3\,000\,t^2}}$

(1) "The allowed WEB-RESISTANCE S_b, in pounds per square inch, computed from this compression-formula. (See, also, page 183.)

(2) "The distance a, or the distance over which the end-reaction must be distributed when the shearing-stress V in the web is the maximum allowable stress of 10 000 lb per sq in.

(3) "The allowable END-REACTION R, when a is taken at 3½ in, which is the usual length of beam actually resting on the 4-in angles ordinarily used in building-construction for beam-seats.

(4) "The allowable SHEAR V, on the gross area of the cross-section of the beam or channel-webs, at 10 000 lb per sq in."

In regard to the second condition of WEB-BUCKLING, the MAXIMUM ALLOWABLE SHEAR may be calculated by the formula,

$$V = \frac{12\,000\,dt}{1 + \dfrac{h^2}{1\,500\,t^2}}$$

in which V = the maximum safe web-shear in pounds; d = the depth of the beam; t = the thickness of the web; and h = the height between the flange-fillets. (See Example 15, this chapter and also example on pages 182 and 183.)

"In addition to these data which have to do with the MAXIMUM LOADS on beams and channels as computed from the WEB-RESISTANCE, Tables II and III give, also, the MAXIMUM BENDING MOMENTS in foot-pounds, obtained by the multiplication of the SECTION-MODULUS of each section by the allowed FIBER-STRESS 16 000 lb per sq in and the division of the product by 12 in order to reduce it to a foot-pound basis. These maximum bending moments may be used on occasion instead of the table of properties to ascertain the proper size of a section to be used in any particular instance."

... of which V = the maximum safe web-shear in pounds; d = the depth of the beam; t = the thickness of web; and h = the distance between the flange-fillets.

It is to be noted that the length of the element in compression on the 45° line is $h\sqrt{2}$, and that the square of this length is $2h^2$. It is this value, $2h^2$, that is substituted for l^2 in the column-formula used by the Cambria Steel Company in deducing its formula for shear based on web-buckling. The tensile stress, however, tends to keep the compressive ... from buckling the web, and for this reason the Passaic and Bethlehem engineers ... the more liberal value of $3\,000\,t^2$ instead of $1\,500\,t^2$. The Passaic Steel Company, however, used the more conservative unit value of 10 000 lb, reduced, instead of the ... lb used by the others. The Passaic and Cambria formulas give about the same ..., a 12-in, 31½-lb I beam by the former having a safe shear of 33 352 lb and by the ..., 33 188 lb.

The Passaic Steel Company is no longer in existence and their handbook is out of print. The Bethlehem Steel Company's handbook has tables for Bethlehem shapes only. If, in any case, no table of maximum shears of beams, based on web-crippling, is at hand, it is suggested that the values may be determined from the formula,

$$V = \frac{12\,000\,dt}{1 + \dfrac{h^2}{1\,500\,t^2}}$$

in which, as before, V = the maximum safe web-shear in pounds; d = the depth of the beam; t = the thickness of the web; and h = the distance between the flange-fillets. For the beam mentioned and used in Example 15, page 571, in this chapter and in the example on pages 182 and 183, d = 12 in, t = 0.35 in, h = 9.762 in, t^2 = 0.1225, ... 95.296644 and V = 33 188 lb. This formula is recommended as being the most conservative, although there is not a great difference in the results, and the formula of the former Passaic Steel Company is retained elsewhere in Kidder's Pocket-Book. See, for example, page 686 and Table III of Chapter XX. Editor-in-chief.

Table VII is a table computed by Mr. Kidder, giving the strength small rectangular steel channels or grooved steel. These are often used supporting metal lath in suspended ceilings, and the table will be found us in determining the size to use for any given span and spacing.

4. Tables of Safe Loads for Steel Beams and Girders. Examp

Example 9. Direct Bending from a Uniformly Distributed Load. As illustration of the use of these tables let it be required to determine the pr size and weight of an I beam to carry safely a uniformly distributed loa 34 000 lb over a span of 20 ft, the weight of the beam not being included.

Solution. From Table IV, page 579, a 15-in 50-lb beam will carry 34 40 The weight of this beam is 50 lb × 20 ft = 1 000 lb, making a total load t supported of 35 000 lb. This is so little in excess of the safe load that the e need not be considered. Had the difference been more, however, the heavier beam should be used.

Example 10. Direct Bending from a Concentrated Load. To illustrate use of the tables to determine the size and weight of beams required to concentrated loads, Examples 10 and 11 are given. What I beam, 15 ft in s will safely support 8 000 lb, concentrated at a point 5 ft from the left sup

Solution. The distance 5 ft is one-third of the span, and the conver factor for this (Table IV, page 632) is 1.78. The equivalent uniformly tributed load, therefore, is 8 000 × 1.78 = 14 240 lb, and from Table IV, 581, a 9-in 25-lb I beam will carry 14 500 lb for a span of 15 ft, and will answer the purpose.

Example 11. Direct Bending from Two Equal Concentrated Loads. I beam, 15 ft in span, will safely support two equal concentrated loads of lb each, applied 5 ft from each end?

Solution. The distance 5 ft is one-third the span, but the multiplier in case is 2⅔ (page 566). Hence, the equivalent uniformly distributed lo 6 000 × 2⅔ = 16 000 lb and the beam required (Table IV, page 580) is a 25.4-lb I beam which will carry 17 400 lb. The same result is obtain using Formula (11)′, page 562. This formula is, $I/c = 12\,Pm/S$. Substit $I/c = 12 \times 6\,000 \times 5/16\,000 = 22.5$. The nearest section-modulus to this is that of a 10-in 25.4-lb I beam.

Example 12. Maximum Bending Moment from a Distributed Load Ove of the Span. The beam in Example 5, Case V, page 561, has a maximum ing moment of 26 666 ft-lb. What beam is required?

Solution. The nearest bending moment to this in the first column of II, page 575, is 27 240 ft-lb, which corresponds to a 9-in 25-lb I beam.

Example 13. Allowable Web-Shear.* The maximum shear in the be Example 12 is just at the right of the left reaction or bearing, and equals lb. Is the beam safe for shear?

Solution. From Table II, page 575, in the column for V, the allowabl shear for a 9-in 25-lb beam is 36 540 lb. Hence, the beam is safe if web-bu is not taken into account.

Example 14. Shear.* It is required to determine the maximum load a 9-in 25-lb I beam can support without exceeding the safe web-resista the section.

* See paragraphs and foot-note relating to buckling of beam-webs, pages 567 t

Solution. From Table IV, page 581, the maximum load for this beam, given in small figures above the heavy, horizontal lines, is 73 100 lb.

Example 15. Safe Buckling Resistance. See, also, paragraphs and foot-note relating to buckling of beam-webs on pages 567 to 569 and also example on pages 182 and 183. According to Table II, page 575, the allowable web-shear for a 12-in, 31.8-lb I beam is 42 000 lb. Will this shear cause the web of the beam to buckle?

Solution. The web-shear is determined by multiplying the area of the web, that is, 0.35 in × 12 in = 4.2 sq in, by 10 000 lb per sq in, the safe unit shearing-stress. The maximum shear which will not cause the web to fail by buckling may be found by the formula given on page 569 for the second condition of web-buckling.

$$V = \frac{12\,000\,dt}{1 + \dfrac{h^2}{1\,500\,t^2}}$$

From the dimensions of structural beams (see Carnegie's Pocket Companion, Beams, Profiles, Weights, etc.) the thickness t of the web of a 12-in, 31.8-lb beam is 0.35 in, the depth of the beam is d = 12 in and h, the distance between flange-fillets, is 9.762 in. Substituting these values in the formula,

$$V = \frac{12\,000 \times 12 \times 0.35}{1 + \dfrac{9.762^2}{1\,500 \times 0.35^2}} = \frac{50\,400}{1 + \dfrac{95.296644}{1\,500 \times 0.1225}} = \frac{50\,400}{1 + \dfrac{95.296644}{183.75}} = \frac{50\,400}{\dfrac{279.046644}{183.75}}$$

$$= \frac{50\,400 \times 183.75}{279.046644} = \frac{9\,261\,000}{279.046644} = 33\,188, \text{ or about } 33\,190 \text{ lb}$$

As this is less than the allowable web-shear of 42 000 lb given in the tables, account is to be taken of the web-buckling from the second condition mentioned in the preceding pages, a larger or heavier beam should be used or the loads reduced, so that the maximum shear will not exceed 33 190 lb. (For determining conditions for web-buckling of steel beams in grillages, based on direct compression, see page 183.)

Example 16. Safe End-Reactions for Web-Buckling. In Example 8, page 5, the two 24-in 90-lb I beams carry 170 000 lb + (4 000 lb, the weight of the beams) = 174 000, lb or 87 000 lb for each beam. Assuming that they rest on 4-in brackets riveted to columns at each end of the span, are the end-reactions excessive?

Solution. Since the loading is symmetrical, each reaction for each beam is one-half the total load on each beam, or 43 500 lb. From the last column in Table II, page 574, the maximum end-reaction R, for a 24-in 90-lb beam, is 310 lb. Hence, the beam is safe as far as the compression from the end-reactions is concerned.

Strut-Beams. It is not considered good construction to subject a strut to a transverse loading, causing a certain amount of flexure in it and thus adding to the compressive stress. Conditions often exist, however, where practical considerations make it desirable to use a strut as a beam, also, as in the top chord or in the principals of a truss. To determine the size of a member in a case of this kind the following method should be used:

(1) Find the section-modulus I/c, for the member for the transverse load by formulas (2)′ to (11)′, using 12 000 lb per sq in as the value of S, and find the area of the cross-section of a steel shape corresponding to the value of I/c thus found. See note at end of Example 17, relating to value of S.

(2) Find the section-area required to resist the compressive stress, by divic that stress by the value opposite l/r in column VIII of Table XI, page 49:

(3) Add together the two areas and use for the required member a piec pieces of material having a section-area next larger than the total area fo

Example 17. Strut-Beam. Combined Bending and Compression. principal rafter in a truss, 8 ft 6 in long between joints, supports the end purlin at the middle of the span. The weight from the purlin is 2 800 lb and compressive stress in the rafter 30 000 lb. It is proposed to use a pair of an for the rafter, set with the long legs vertical and ½ in apart. What are dimensions of the angles, the strut being braced laterally?

Solution. (1) By Formula (8)′, $I/c = 3 \times 2\,800 \times 8.5/12\,000 = 5.95$ for pair of angles, or 2.98 for each angle. (See note at end of this example.) F Table XI, page 363, the nearest value to this with reference to the axis 1- 3.0, the section-modulus for a 5 by 3½ by ½-in angle. The section-area of angle is 4 sq in and of two angles, 8 sq in.

(2) From Table XVI, page 371, the least r for a pair of 5 by 3½ by angles, which would be about the axis 1–1, since the strut is braced later is about 1.58 (between 1.53 and 1.61). Then the slenderness-ratio $l/r =$ 6 in/1.58 in = 102 in/1.58 in = 64.5. From column VIII, Table XI, 493, $S = 9\,250$ lb per sq in. Hence, 30 000 lb/9 250 lb per sq in = 3.24 sq approximately.

(3) The section-area required, therefore, is 8 + 3.24 = 11.24 sq in, which, Table XI, page 363, is about equivalent to that of two 5 by 4 by 1½ angles. As the section-area in both calculations exceeds that actually requ no allowance for the weight of the angles need be made.

Note. Because of the increase in the tendency of the strut to deflect, ca by the combined stresses of flexure and compression, lower values of S are than in the cases of simple flexure, or of simple compression.

Tie-Beams. Steel beams subject to combined tensile and transverse str should be calculated in a way similar to that explained above for strut-be The section necessary to resist the transverse stress should be found first, the section-area necessary to resist the tensile stress, and the two added toge

Example 18. Tie-Beam. Combined Bending and Tension. One span tie-beam, 10 ft between joints, supports a load of 6 000 lb at the middle, a the same time is under a tensile stress of 84 000 lb. It is proposed to use steel channels for the tie-beam. What size and weight are required fo channels?

Solution. A load of 6 000 lb applied at the middle of a beam has the effect as a load of 12 000 lb uniformly distributed, or 6 000 lb for each cha From Table V, page 584, a 7-in, 9.8-lb channel will be required, its sec area (Table VIII, page 359) being 2.85 sq in. The additional area requir resist the tensile stress is 84 000 lb/16 000 lb per sq in = 5.25 sq in, or 2.6 each channel. The total area for each channel, therefore, should be 2.85 - = 5.48 sq in. A 7-in, 19.75-lb channel has a section-area of 5.79 sq in an 8-in, 18.75-lb channel has a section-area of 5.49 sq in. Either one w sufficient, but the 8-in channel will probably be more economical, as it w 1 lb per ft less.

Example 19. Channel, Set Flatwise. What is the size of the channe flatwise, required to support a uniformly distributed load of 180 lb per ft a span of 10 ft, or 120 in?

Solution. $W = 180 \times 10 = 1\,800$ lb. From Case V, page 326, $M_{max} = W$ $1\,800 \times 120/8 = 27\,000$ in-lb. From Formula (3)′, page 557, $I/c = M$

.000/16 000 = 1.7. From Table VIII, page 359, the I/c about the axis 2-2 ...responding to this is that of a 12-in, 20.7-lb channel.

...ample 20. Rectangular Steel Bar with Long Side Vertical. In a suspended, ...stered ceiling it is proposed to use 2 by ⅝-in steel bars, 4 ft or 48 in long, to ...ry the plaster. What is the safe load each bar will support, if set with the ...g side vertical?

...lution. From Table I, page 346, the I for a 2 by ⅝-in bar is 0.250. c = ...-half the depth = 1 in. $I/c = 0.250/1 = 0.250$. Also, from Formula (2)', ...e 557, $M_{max} = SI/c$. Substituting, $M_{max} = 16\,000 \times 0.250 = 4\,000$ in-lb ...t, from Case V, page 326, $M_{max} = Wl/8$, and hence, $4\,000 = W \times 48/8 =$ ...W, and $W = 4\,000/6 = 666$ lb.

Oblique Loading of I Beams and Channels *

Oblique Loading of Purlins on Sloping Roofs. (See, also pages 593, ...9 and 1170.) In Tables II to V it is assumed that I beams and channels ...set with webs vertical and carry vertical loads. This is not the case when ...d as PURLINS ON SLOPING ROOFS. There are then fiber-stresses due to the ...ponents of the bending moment both at right-angles and parallel to the ...e of the roof. The resultant fiber-stress may be calculated from the ...ation on page 1170. This equation is used in determining the values ...n in Table I A. It may be noted that the second term causes the fiber-...s to increase rapidly with the slope of the roof. If purlins were propor-...ed according to the equation given or from the Table I A, they would often ...uch larger than those commonly used. For small slopes the second term ...the equation may be reduced or eliminated by the stiffness of the roof-...ing, and for other slopes by connecting the purlins with SAG-RODS running ...the sloping sides of the roof to an unyielding connection at the peak.

...e I A. Ratio of Maximum Fiber-Stress to Bending Moment for I-Beam and Channel Purlins Set of Right-Angles to Rafters and Free to Move in Any Direction. Loading Vert'cal and Oblique to Web

Purlin	Slope of roof in inches per foot †						
	0	1	2	4	5	6	8
...in I beam 12.5 lb	0.14	0.21	0.28	0.42	0.47	0.53	0.61
...in I beam 15.3 lb	0.10	0.15	0.21	0.31	0.35	0.39	0.46
...in I beam 18.4 lb	0.07	0.11	0.16	0.23	0.27	0.30	0.35
...in I beam 21.8 lb	0.05	0.09	0.12	0.18	0.21	0.24	0.28
...in I beam 25.4 lb	0.04	0.07	0.10	0.15	0.17	0.19	0.22
...in I beam 31.8 lb	0.03	0.05	0.07	0.11	0.13	0.14	0.17
...in channel 8.2 lb	0.23	0.40	0.56	0.85	0.98	1.10	1.30
...in channel 9.8 lb	0.17	0.30	0.42	0.66	0.76	0.85	1.01
...in channel 11.5 lb	0.12	0.23	0.33	0.52	0.60	0.68	0.80
...in channel 13.4 lb	0.10	0.18	0.26	0.42	0.48	0.55	0.65
...in channel 15.3 lb	0.08	0.15	0.21	0.34	0.40	0.45	0.54
...in channel 20.7 lb	0.05	0.09	0.14	0.23	0.26	0.30	0.36

* ...m Notes by Robins Fleming.
† ...alues vary slightly for slight variations in weights and section-areas of beams and ...nels.

Table II.* † Maximum Bending Moments and Web-Resistance of I Be

M_{max}	d	w	t	V	S_b†	a	R
Maximum bending moment	Depth of beam	Weight per lin ft	Thickness of web	Allowable web-shear	Allowable buckling resistance	Minimum end-bearing	End reacti $a=3½$
ft-lb	in	lb	in	lb	lb per sq in	in	lb
292 130	27	90.0	0.524	141 480	10 080	20.0	54 1
328 390		115.0	0.737	180 000	13 460	11.8	95 8
320 390		110.0	0.675	165 120	12 960	12.5	84 6
312 390		105.0	0.625	150 000	12 350	13.4	73 3
264 400		100.0	0.747	180 960	13 490	11.8	96 6
256 560	24	95.0	0.686	166 320	13 000	12.5	85 6
248 710		90.0	0.624	151 440	12 410	13.3	74 4
240 870		85.0	0.563	136 800	11 710	14 5	63 4
231 920		80.0	0.500	120 000	10 690	16.5	50 7
216 670		74.2	0.476	114 240	10 260	17.4	46 4
156 930	21	60.4	0.428	89 880	10 500	14.8	39 3
220 750		100.0	0.873	176 800	15 080	8.3	113
214 210		95.0	0.800	162 000	14 720	8.6	101
207 680		90.0	0.726	147 400	14 300	9.0	89
201 140		85.0	0.653	132 600	13 780	9.5	77
195 510	20	80.4	0.600	120 000	13 230	10.1	67
169 170		75.0	0.641	129 800	13 660	9.6	75
162 640		70.0	0.567	115 000	12 980	10.4	63
155 930		65.4	0.500	100 000	12 080	11.6	51
186 720		90.0	0.796	145 260	15 140	7.4	97
180 840		85.0	0.714	130 500	14 700	7.7	85
174 960		80.0	0.632	115 920	14 160	8.2	72
169 080		75.6	0.560	101 160	13 450	8.9	60
136 480	18	70.0	0.711	129 420	14 670	7.8	84
130 590		65.0	0.629	114 660	14 110	8.3	71
124 710		60.0	0.547	99 900	13 380	9.0	59
117 860		55.0	0.460	82 800	12 220	10.2	44
109 200		48.2	0.380	68 400	10 800	12.2	32
122 890		75.0	0.868	132 300	16 050	5.6	*102
117 980		70.0	0.770	117 600	15 690	5.8	89
113 080		65.0	0.672	102 900	15 210	6.1	75
108 270		60.8	0.590	88 500	14 600	6.5	62
90 850	15	55.0	0.648	98 400	15 040	6.2	71
85 940		50.0	0.550	83 700	14 340	6.7	58
81 040		45.0	0.452	69 000	13 350	7.5	44
78 530		42.9	0.410	61 500	12 670	8.1	37
72 130		37.5	0.332	49 800	11 180	9.7	26

V is computed at 10 000 lb per sq in of gross area of web-section.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

† See, also, foot-note on page 568, with paragraphs relating to this table and Tab III, and paragraphs on page 567, relating to web-buckling of steel beams. See page 181.

[illegible]le II * † (Continued). Maximum Bending Moments and Web-Resistance of I Beams

M_{max}	d	w	t	V	S_b †	n	R
Maximum bending moment	Depth of beam	Weight per lin ft	Thickness of web	Allowable web-shear	Allowable buckling resistance	Minimum end-bearing	End-reaction $a = 3\frac{1}{2}$ in
ft-lb	in	lb	in	lb	lb per sq in	in	lb
71 330		55.0	0 810	98 520	16 470	[illegible]	[illegible]
67 410		50 0	0.687	83 880	16 030		
63 490		45 0	0.565	69 120	15 390		
59 770	12	40 8	0.460	55 200	14 480		
50 730		35 0	0 428	52 320	14 230		
47 360		31 8	0.350	42 000	13 060		
44 270		27.9	0.284	34 080	11 680		
42 310		40 0	0 741	74 900	16 690		
39 050		35 0	0 594	60 200	16 120		
35 780	10	30 0	0.447	45 500	15 190		
31 560		25 4	0.310	31 000	13 410		
28 270		22.4	0.252	25 200	12 130		
33 120		35 0	0 734	65 880	16 870		
30 150		30 0	0 561	51 210	16 260		
27 240	9	25.0	0.397	36 540	15 160		
25 160		21.8	0 290	26 100	13 620		
22 810		25.5	0 532	43 280	16 440		
21 500		23 0	0 441	35 920	15 910		
19 190	8	20 5	0 349	28 560	15 120		
16 960		18 4	0 270	21 600	13 870		
15 470		17 5	0.220	17 600	12 700		
16 270		20.0	0 450	32 060	16 350		
14 330	7	17.5	0.345	24 710	15 570		
12 800		15 3	0 250	17 500	14 150		
11 640		17.25	0 465	28 500	16 810		
10 660	6	14 75	0 343	21 120	16 050		
9 880		12 5	0.230	13 800	14 480		
8 680		14 75	0 494	25 200	17 280		
7 360	5	12 25	0 347	17 850	16 580		
6 450		10.00	0 210	10 500	14 870		
[illegible]4 760		10.5	0.400	16 400	17 510		
[illegible]4 500	4	9 5	0.326	13 480	16 940		
[illegible]4 240		8.5	0 253	10 520	16 360		
[illegible]3 960		7.7	0.190	7 600	15 360		
[illegible]2 590		7 5	0.349	10 830	17 560		
[illegible]2 350	3	6.5	0.251	7 890	17 020		
[illegible]2 110		5.7	0.170	5 100	15 950		

V is computed at 10 000 lb per sq in of gross area of web-section.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

† See also, foot-note on page 568, with paragraphs relating to this table and to Tabl[e] [illegible] paragraphs on page 567, relating to web-buckling of steel beams. See, als[o] [illegible] p 603.

Table III.*† Maximum Bending Moments and Web-Resistances of Chann

M_{max}	d	w	t	V	S_b†	a	R
Maximum bending moment	Depth of channel	Weight per lin ft	Thickness of web	Allowable web-shear	Allowable buckling resistance	Minimum end-bearing	En reacti a=3½
ft-lb	in	lb	in	lb	lb per sq in	in	lb
76 490	15	55.0	0.814	122 700	15 820	5.7	93 8
71 590		50.0	0.716	108 000	15 390	6.0	80 3
66 680		45.0	0.618	93 300	14 820	6.4	66 8
61 780		40.0	0.520	78 600	14 040	6.9	53 3
56 880		35.0	0.422	63 900	12 900	7.9	39 8
55 570		33.9	0.400	60 000	12 510	8.2	36 2
64 360	13	50.0	0.787	102 830	16 150	4.8	86 2
60 110		45.0	0.673	88 140	15 630	5.0	71 7
55 870		40.0	0.560	73 450	15 020	5.4	57 2
53 320		37.0	0.492	64 610	14 470	5.7	48 5
51 620		35.0	0.447	58 760	14 020	6.0	42 7
48 740		31.8	0.375	48 750	13 000	6.8	32 9
43 760	12	40.0	0.755	90 960	16 260	4.4	80 0
39 840		35.0	0.632	76 320	15 730	4.6	65 0
35 920		30.0	0.510	61 560	14 950	5.0	49 8
32 000		25.0	0.387	46 800	13 670	5.8	34 6
28 470		20.7	0.280	33 600	11 570	7.4	21 0
30 800	10	35.0	0.820	82 300	16 900	3.4	83 4
27 530		30.0	0.673	67 600	16 440	3.6	66 6
24 260		25.0	0.526	52 900	15 730	3.9	49
20 990		20.0	0.379	38 200	14 470	4.4	33 1
17 840		15.3	0.240	24 000	11 780	6.0	16
20 950	9	25.0	0.612	55 350	16 470	3.2	58
18 010		20.0	0.448	40 680	15 550	3.5	40
15 070		15.0	0.285	25 920	13 590	4.4	22
14 080		13.4	0.230	20 700	12 220	5.1	16
15 920	8	21.25	0.579	46 560	16 620	2.8	53
14 610		18.75	0.487	39 200	16 170	2.9	43
13 310		16.25	0.395	31 920	15 530	3.2	34
12 000		13.75	0.303	24 560	14 490	3.5	24
10 770		11.50	0.220	17 600	12 700	4.3	15
12 640	7	19.75	0.629	44 310	17 090	2.3	56
11 490		17.25	0.524	36 960	16 700	2.4	46
10 350		14.75	0.419	29 610	16 130	2.6	35
9 210		12.25	0.314	22 260	15 190	2.9	25
8 030		9.80	0.210	14 700	13 230	3.5	14
8 680	6	15.5	0.559	33 780	17 150	2.0	48
7 700		13.0	0.437	26 400	16 640	2.1	36
6 720		10.5	0.314	19 080	15 730	2.3	25
5 780		8.2	0.200	12 000	13 810	2.8	13
5 550	5	11.5	0.472	23 850	17 180	1.7	38
4 730		9.0	0.325	16 500	16 380	1.8	25
3 960		6.7	0.190	9 500	14 450	2.2	13
3 050	4	7.25	0.320	13 000	16 870	1.4	24
2 790		6.25	0.247	10 080	16 250	1.5	18
2 530		5.40	0.180	7 200	15 150	1.6	12
1 840	3	6.0	0.356	10 860	17 560	1.0	27
1 640		5.0	0.258	7 920	17 030	1.0	19
1 450		4.1	0.170	5 100	15 940	1.1	11

V is computed at 10 000 lb per sq in of gross area of web-section.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

† See, also, foot-note on page 568, with paragraphs relating to this table and to III, and paragraphs on page 567, relating to web-buckling of steel beams. Se page 183.

Table IV.* Safe Uniform Loads in Units of 1 000 Pounds for Steel I Beams

[Max]imum bending stress, 16 000 lb per sq in. Beams secured against yielding sidewi[se]

Span, ft	27-in 90 lb	24-in 115 lb	24-in 110 lb	24-in 105.9 lb	24-in 100 lb	24-in 95 lb	24-in 90 lb	24-in 85 lb	24-in 79.9 lb	24-in 74.2 lb	21-in 60.4 lb	Coefficient of deflection *
......	...	...	...	...	[illegible]	...	...	...	...	...	...	......
6	...	...	...	...	352.5	[illegible]	[illegible]	...	...	...	170.8	0.60
7	...	[illegible]	[illegible]	...	302.2	293.2	284.2	[illegible]	[illegible]	228.5	179.3	0.81
8	[illegible]	328.4	320.4	[illegible]	264.4	256.6	248.7	240.9	[illegible]	216.7	156.9	1.06
9	259.3	291.9	284.8	277.7	235.0	228.0	221.1	214.1	206.1	192.6	139.5	1.34
10	233.4	262.7	256.3	249.9	211.5	205.2	199.0	192.7	185.5	173.3	125.5	1.66
11	212.2	238.8	233.0	227.2	192.3	186.6	180.9	175.2	168.7	157.6	114.1	2.00
12	194.5	218.9	213.6	208.3	176.3	171.0	165.8	160.6	154.6	144.5	104.6	2.38
13	179.5	202.1	197.2	192.2	162.7	157.9	153.1	148.2	142.7	133.3	96.5	2.80
14	166.7	187.7	183.1	178.5	151.1	146.6	142.1	137.6	132.5	123.8	89.7	3.24
15	156.6	175.1	170.9	166.6	141.0	136.8	132.6	128.5	123.7	115.6	83.7	3.72
16	145.9	164.2	160.2	156.2	132.2	128.3	124.4	120.4	116.0	108.3	78.4	4.24
17	137.3	154.5	150.8	147.0	124.4	120.7	117.0	113.4	109.1	102.0	73.8	4.78
18	129.7	146.0	142.4	138.8	117.5	114.0	110.5	107.1	103.1	96.3	69.7	5.36
19	122.8	138.3	134.9	131.5	111.3	108.0	104.7	101.4	97.6	91.2	66.1	5.98
20	116.7	131.4	128.2	125.0	105.8	102.6	99.5	96.3	92.8	86.7	62.8	6.62
21	111.1	125.1	122.1	119.0	100.7	97.7	94.7	91.8	88.3	82.5	59.8	7.30
22	106.1	119.4	116.5	113.6	96.1	93.3	90.4	87.6	84.3	78.8	57.1	8.01
23	101.5	114.2	111.4	108.7	92.0	89.2	86.5	83.8	80.7	75.4	54.6	8.76
24	97.3	109.5	106.8	104.1	88.1	85.5	82.9	80.3	77.3	72.2	52.3	9.53
25	93.4	105.1	102.5	100.0	84.6	82.1	79.6	77.1	74.2	69.3	50.2	10.35
26	89.8	101.0	98.6	96.1	81.4	78.9	76.5	74.1	71.4	66.7	48.3	11.19
27	86.4	97.3	94.9	92.6	78.3	76.0	73.7	71.4	68.7	64.2	46.5	12.07
28	83.4	93.8	91.5	89.3	75.5	73.3	71.1	68.8	66.3	61.9	44.8	12.98
29	80.5	90.6	88.4	86.2	72.9	70.8	68.6	66.4	64.0	59.8	43.3	13.92
30	77.8	87.6	85.4	83.3	70.5	68.4	66.3	64.2	61.8	57.8	41.8	14.90
31	75.3	84.7	82.7	80.6	68.2	66.2	64.2	62.2	59.8	55.9	40.5	15.91
32	72.9	82.1	80.1	78.1	66.1	64.1	62.2	60.2	58.0	54.2	39.2	16.95
33	70.7	79.6	77.7	75.7	64.1	62.2	60.3	58.4	56.2	52.5	38.0	18.03
34	68.6	77.3	75.4	73.5	62.2	60.4	58.5	56.7	54.6	51.0	36.9	19.13
35	66.7	75.1	73.2	71.4	60.4	58.6	56.8	55.1	53.0	49.5	35.9	20.28
36	64.8	73.0	71.2	69.4	58.8	57.0	55.3	53.5	51.5	48.2	34.9	21.45
37	63.1	71.0	69.3	67.5	57.2	55.5	53.8	52.1	50.1	46.8	33.9	22.66
38	61.4	69.1	67.5	65.8	55.7	54.0	52.4	50.7	48.8	45.6	33.0	23.90
39	59.8	67.4	65.7	64.1	54.2	52.6	51.0	49.4	47.6	44.4	32.2	25.18
40	58.4	65.7	64.1	62.5	52.9	51.3	49.7	48.2	46.4	43.3	31.4	26.48
41	56.9	64.1	62.5	61.0	51.6	50.1	48.5	47.0	45.3	42.3	30.6	27.82
42	55.6	62.6	61.0	59.5	50.4	48.9	47.4	45.9	44.2	41.3	29.9	29.20
43	54.3	61.1	59.6	58.1	49.2	47.7	46.3	44.8	43.1	40.3	29.2	30.60
44	53.0	59.7	58.3	56.8	48.1	46.6	45.2	43.8	42.2	39.4	28.5	32.04
45	51.9	58.4	57.0	55.5	47.0	45.6	44.2	42.8	41.2	38.5	...	33.52
46	50.7	57.1	55.7	54.3	46.0	44.6	43.3	41.9	40.3	37.7	...	35.02
47	49.7	55.9	54.1	53.2	45.0	43.7	42.3	41.0	39.5	36.9	...	36.56
48	48.6	54.7	53.4	52.1	44.1	42.8	41.5	40.1	38.7	36.1	...	38.14
49	47.6	53.6	52.3	51.0	43.2	41.9	40.6	39.3	37.9	35.4	...	39.74
50	46.7	52.5	51.3	50.0	42.3	41.0	39.8	38.5	37.1	34.7	...	41.38

Loads above the upper heavy lines will cause maximum allowable shears in [we]bs. See, also, paragraphs in text and foot-note with same, page 567, relating to [we]b-buckling in beams

[Lo]ads below the lower broken lines will cause excessive deflections

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table IV * (Continued). Safe Uniform Loads in Units of 1 000 Pounds for Steel I Beams

Maximum bending stress, 16 000 lb per sq in. Beams secured against yielding side

Span, ft	Depth and weight of sections												Co cie de ti
	20-in								18-in				
	100 lb	95 lb	90 lb	85 lb	81.4 lb	75 lb	70 lb	65.4 lb	90 lb	85 lb	80 lb	75.6 lb	
....	353.6	...	...	...	...	...	...	...	...	...	...	...	...
5	353.2	...	...	...	...	...	...	...	...	...	...	...	0.
....	....	324.0	294.8	...	...	259.6	230.0	...	200.5	261.0	...	...	...
6	294.3	285.6	276.9	265.2	240.0	225.6	216.8	200.0	249.0	241.1	231.8	208.3	0.
7	252.3	244.8	237.7	229.9	223.4	193.3	185.9	178.2	213.4	206.7	200.0	193.2	0.
8	220.7	214.2	207.7	201.1	195.5	169.2	162.6	155.9	186.7	180.8	175.0	169.1	1.
9	196.2	190.4	184.6	178.8	173.8	150.4	144.6	138.6	166.0	160.7	155.5	150.3	1.
10	176.6	171.4	166.1	160.9	156.4	135.3	130.1	124.7	149.4	144.7	140.0	135.3	1.
11	160.5	155.8	151.0	146.3	142.2	123.0	118.3	113.4	135.8	131.5	127.2	123.0	2
12	147.2	142.8	138.5	134.1	130.3	112.8	108.4	104.0	124.5	120.6	116.6	112.7	2
13	135.8	131.8	127.8	123.8	120.3	104.1	100.1	96.0	114.9	111.3	107.7	104.1	2
14	126.1	122.4	118.7	114.9	111.7	96.7	92.9	89.1	106.7	103.3	100.0	96.6	3
15	117.7	114.2	110.8	107.3	104.3	90.2	86.7	83.2	99.6	96.4	93.3	90.2	3
16	110.4	107.1	103.8	100.6	97.7	84.6	81.3	78.0	93.4	90.4	87.5	84.5	4
17	103.9	100.8	97.7	94.1	92.0	79.6	76.5	73.4	87.9	85.1	82.3	79.6	4
18	98.1	95.2	92.3	89.4	86.9	76.3	72.3	69.3	83.0	80.4	77.8	75.1	5
19	92.9	90.2	87.4	84.7	82.3	71.2	68.5	65.7	78.6	76.1	73.7	71.2	5
20	88.3	85.7	83.1	80.5	78.2	67.7	65.1	62.4	74.7	72.3	70.0	67.6	6
21	84.1	81.6	79.1	76.6	74.5	64.4	62.0	59.4	71.1	68.9	66.7	64.4	7
22	80.3	77.9	75.5	73.1	71.1	61.5	59.1	56.7	67.9	65.8	63.6	61.5	[illegible]
23	76.8	74.5	72.2	70.0	68.0	58.8	56.6	54.2	64.9	62.9	60.9	58.8	[illegible]
24	73.6	71.4	69.2	67.0	65.2	56.4	54.2	52.0	62.2	60.3	58.3	56.4	[illegible]
25	70.6	68.5	66.5	64.4	62.6	54.1	52.0	49.9	59.8	57.9	56.0	54.1	10
26	67.9	65.9	63.9	61.9	60.2	52.1	50.0	48.0	57.5	55.6	53.8	52.0	11
27	65.4	63.5	61.5	59.6	57.9	50.1	48.2	46.2	55.3	53.6	51.8	50.1	1[illegible]
28	63.1	61.2	59.3	57.5	55.9	48.3	46.5	44.6	53.3	51.7	50.0	48.3	1[illegible]
29	60.9	59.1	57.3	55.5	53.9	46.7	44.9	43.0	51.5	49.9	48.3	46.6	1[illegible]
30	58.9	57.1	55.4	53.6	52.1	45.1	43.4	41.6	49.8	48.2	46.7	45.1	1[illegible]
31	57.0	55.3	53.6	51.9	50.5	43.7	42.0	40.2	48.2	46.7	45.2	43.6	1[illegible]
32	55.2	53.6	51.9	50.3	48.9	42.3	40.7	39.0	46.7	45.2	43.7	42.3	1[illegible]
33	53.5	51.9	50.4	48.8	47.4	41.0	39.4	37.8	45.3	43.8	42.4	41.0	1[illegible]
34	51.9	50.4	48.9	47.3	46.0	39.8	38.3	36.7	43.9	42.6	41.2	39.8	1[illegible]
35	50.5	49.0	47.5	46.0	44.7	38.7	37.2	35.6	42.7	41.3	40.0	38.6	2[illegible]
36	49.1	47.6	46.2	44.7	43.4	37.6	36.1	34.7	41.5	40.2	38.9	37.6	2[illegible]
37	47.7	46.3	44.9	43.5	42.3	36.6	35.2	33.7	40.4	39.1	37.8	36.6	2[illegible]
38	46.5	45.1	43.7	42.3	41.2	35.6	34.2	32.8	39.3	38.1	36.8	35.6	2[illegible]
39	45.3	43.9	42.6	41.3	40.1	34.7	33.4	32.0	...	...	...	...	2[illegible]
40	44.1	42.8	41.5	40.2	39.1	33.8	32.5	31.2	...	...	...	...	2[illegible]
41	43.1	41.8	40.5	39.2	38.1	33.0	31.7	30.4	...	...	...	...	[illegible]
42	42.0	40.8	39.6	38.3	37.2	32.2	31.0	29.7	...	...	...	...	[illegible]

Loads above the upper heavy lines will cause maximum allowable she webs. See, also, paragraphs in text and foot-note with same, page 567, re to web-buckling in beams

Loads below the lower broken lines will cause excessive deflections

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table IV * (Continued). Safe Uniform Loads in Units of 1 000 Pound for Steel I Beams

imum bending stress, 16 000 lb per sq in. Beams secured against yielding si

Span, ft	Depth and weight of sections												
	18-in					15-in							
	70 lb	65 lb	60 lb	54.7 lb	48.2 lb	75 lb	70 lb	65 lb	60.8 lb	55 lb	50 lb	45 lb	42.9 lb
..	...	...	...	...	...	261.6	...	...	...	196.8	...	...	...
4	258.8	229.3	199.8	...	...	245.8	235.2	205.8	177.0	181.7	167.4	148.0	...
5	218.4	208.9	199.5	...	...	196.6	188.8	180.9	173.2	145.4	137.5	129.7	...
..	...	...	...	165.6	...	...	...	...	...	...	...	...	121.0
6	182.0	174.1	166.3	157.1	136.8	163.8	157.3	150.8	144.4	121.1	114.6	108.1	104.7
7	156.0	149.2	142.5	134.7	124.8	140.4	134.8	129.2	123.7	103.8	98.2	92.6	89.8
8	136.5	130.6	124.7	117.9	109.2	122.9	118.0	113.1	108.3	90.8	85.9	81.0	78.5
9	121.3	116.1	110.9	104.8	97.1	109.2	104.9	100.5	96.2	80.8	76.4	72.0	69.8
10	109.2	104.5	99.8	94.3	87.4	98.3	94.4	90.5	86.6	72.7	68.8	64.8	62.8
11	99.3	95.0	90.7	85.7	79.4	89.4	85.8	82.2	78.7	66.1	62.5	58.9	57.1
12	91.0	87.1	83.1	78.6	72.8	81.9	78.7	75.4	72.2	60.6	57.3	54.0	52.4
13	84.0	80.4	76.7	72.5	67.2	75.6	72.6	69.6	66.6	55.9	52.9	49.9	48.3
14	78.0	74.6	71.3	67.3	62.4	70.2	67.4	64.6	61.9	51.9	49.1	46.3	44.9
15	72.8	69.6	66.5	62.9	58.2	65.5	62.9	60.3	57.7	48.5	45.8	43.2	41.9
16	68.2	65.3	62.4	58.9	54.6	61.4	59.0	56.5	54.1	45.4	43.0	40.5	39.3
17	64.2	61.5	58.7	55.5	51.4	57.8	55.5	53.2	50.9	42.8	40.4	38.1	37.0
18	60.7	58.0	55.4	52.4	48.5	54.6	52.4	50.3	48.1	40.4	38.2	36.0	34.9
19	57.5	55.0	52.5	49.6	46.0	51.7	49.7	47.6	45.6	38.3	36.2	34.1	33.1
20	54.6	52.2	49.9	47.1	43.7	49.2	47.2	45.2	43.3	36.3	34.4	32.4	31.4
21	52.0	49.7	47.5	44.9	41.6	46.8	44.9	43.1	41.2	34.6	32.7	30.9	29.9
22	49.6	47.5	45.3	42.9	39.7	44.7	42.9	41.1	39.4	33.0	31.3	29.5	28.6
23	47.5	45.4	43.4	41.0	38.0	42.7	41.0	39.3	37.7	31.6	29.9	28.2	27.3
24	45.5	43.5	41.6	39.3	36.4	41.0	39.3	37.7	36.1	30.3	28.6	27.0	26.2
25	43.7	41.8	39.9	37.7	34.9	39.3	37.8	36.2	34.6	29.1	27.5	25.9	25.1
26	42.0	40.2	38.4	36.3	33.6	37.8	36.3	34.8	33.3	28.0	26.4	24.9	24.2
27	40.4	38.7	37.0	34.9	32.4	36.4	35.0	33.5	32.1	26.9	25.5	24.0	23.3
28	39.0	37.3	35.6	33.7	31.2	35.1	33.7	32.3	30.9	26.0	24.6	23.2	22.4
29	37.6	36.0	34.4	32.5	30.1	33.9	32.5	31.2	29.9	25.1	23.7	22.4	21.7
30	36.4	34.8	33.3	31.4	29.1	32.8	31.5	30.2	28.9	24.2	22.9	21.6	20.9
31	35.2	33.7	32.2	30.4	28.2	31.7	30.4	29.2	27.9	23.4	22.2	20.9	20.3
32	34.1	32.6	31.2	29.5	27.3	30.7	29.5	28.3	27.1	22.7	21.5	20.3	19.6
33	33.1	31.7	30.2	28.6	26.5	...	...	...	...	...	...	...	...
34	32.1	30.7	29.3	27.7	25.7	...	...	...	...	...	...	...	...
35	31.2	29.8	28.5	26.9	25.0	...	...	...	...	...	...	...	...
36	30.3	29.0	27.7	26.2	24.3	...	...	...	...	...	...	...	...
37	29.5	28.2	27.0	25.5	23.6	...	...	...	...	...	...	...	...
38	28.7	27.5	26.3	24.8	23.0	...	...	...	...	...	...	...	...

Loads above the upper heavy lines will cause maximum allowable shea
webs. See, also, paragraphs in text and foot-note with same, page 567, rel
n web-buckling in beams

Loads below the lower broken lines will cause excessive deflections

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table IV * (Continued). Safe Uniform Loads in Units of 1 000 Pounds for Steel I Beams

Maximum bending stress, 16 000 lb per sq in. Beams secured against yielding sid

Span, ft	Depth and weight of sections													
	15-in	12-in							10-in					
	37.3 lb	55 lb	50 lb	45 lb	40.8 lb	35 lb	31.8 lb	27.9 lb	40 lb	35 lb	30 lb	25.4 lb	22.4 lb
..	...	197.0	...	...	...	...	...	...	140.8	120.4	...	...	...
3	...	190.2	167.9	133.2	...	114.6	...	...	112.8	104.1	91.0	...	...
4	...	142.7	134.8	127.0	110.4	101.5	84.0	...	84.6	78.1	71.6	62.0	...
5	...	114.1	107.9	101.6	95.6	81.2	76.7	...	67.7	62.5	57.2	52.1	50.4
..	90.6	...	...	...	...	...	...	68.2	...	...	...	...	48.5
6	96.1	95.1	89.9	84.7	79.7	67.6	63.9	59.1	56.4	52.1	47.7	43.4	40.4
7	82.4	81.5	77.0	72.6	68.3	58.0	54.8	50.6	48.4	44.6	40.9	37.2	34.6
8	72.1	71.3	67.4	63.5	59.8	50.7	48.0	44.3	42.3	39.0	35.8	32.6	30.3
9	64.1	63.4	59.9	56.4	53.1	45.1	42.6	39.4	37.6	34.7	31.8	28.9	26.9
10	57.7	57.1	53.9	50.8	47.8	40.6	38.4	35.5	33.9	31.2	28.6	26.0	24.2
11	52.4	51.9	49.0	46.2	43.5	36.9	34.9	32.2	30.8	28.4	26.0	23.7	22.0
12	48.1	47.6	44.9	42.3	39.8	33.8	32.0	29.5	28.2	26.0	23.9	21.7	20.2
13	44.4	43.9	41.5	39.1	36.8	31.2	29.5	27.3	26.0	24.0	22.0	20.0	18.6
14	41.2	40.8	38.5	36.3	34.2	29.0	27.4	25.3	24.2	22.3	20.4	18.6	17.3
15	38.4	38.0	36.0	33.9	31.9	27.1	25.6	23.6	22.6	20.8	19.1	17.4	16.2
16	36.0	35.7	33.7	31.7	29.9	25.4	24.0	22.2	21.2	19.5	17.9	16.3	15.1
17	33.9	33.6	31.7	29.9	28.1	23.9	22.6	20.9	19.9	18.4	16.8	15.3	14.3
18	32.0	31.7	30.0	28.2	26.6	22.5	21.3	19.7	18.8	17.4	15.9	14.5	13.5
19	30.4	30.0	28.4	26.7	25.2	21.4	20.2	18.7	17.8	16.4	15.1	13.7	12.8
20	28.8	28.5	27.0	25.4	23.9	20.3	19.2	17.7	16.9	15.6	14.3	13.0	12.1
21	27.5	27.2	25.7	24.2	22.8	19.3	18.3	16.9	16.1	14.9	13.6	12.4	11.5
22	26.2	25.9	24.5	23.1	21.7	18.4	17.4	16.1	15.4	14.2	13.0	11.8	11.0
23	25.1	24.8	23.4	22.1	20.8	17.6	16.7	15.4	...	...	...	...	...
24	24.0	23.8	22.5	21.2	19.9	16.9	16.0	14.8	...	...	...	...	...
25	23.1	22.8	21.6	20.3	19.1	16.2	15.3	14.2	...	...	...	...	...
26	22.2	21.9	20.7	19.5	18.4	15.6	14.8	13.6	...	...	...	...	...
27	21.4	...	...	...	...	...	...	...	...	...	...	...	...
28	20.6	...	...	...	...	...	...	...	...	...	...	...	...
29	19.9	...	...	...	...	...	...	...	...	...	...	...	...
30	19.2	...	...	...	...	...	...	...	...	...	...	...	...
31	18.6	...	...	...	...	...	...	...	...	...	...	...	...
32	18.0	...	...	...	...	...	...	...	...	...	...	...	...

Loads above the upper heavy lines will cause maximum allowable shea webs. See, also, paragraphs in text and foot-note with same, page 567, rel to web-buckling in beams.

Loads below the lower broken lines will cause excessive deflections

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table IV * (Continued). Safe Uniform Loads in Units of 1 000 Pounds for Steel I Beams

Maximum bending stress 16 000 lb per sq in. Beams secured against yielding sidewise

Span, ft	Depth and weight of sections												Coefficient of deflection
	9-in				8-in					7-in			
	35 lb	30 lb	25 lb	21.8 lb	25.5 lb	23 lb	20.5 lb	18.4 lb	17.5 lb	20 lb	17.75 lb	15.3 lb	
....	131.8	102.4	73.1	...	84.6	71.8	57.1	...	...	61.1	49.4	...	......
3	88.3	80.5	72.6	52.2	60.8	57.3	53.9	43.2	...	42.9	39.8	35.0	0.15
4	66.2	60.4	54.5	50.3	45.6	43.0	40.4	37.9	35.2	32.1	29.9	27.6	0.27
5	53.0	48.3	43.6	40.3	36.5	34.4	32.3	30.3	31.1	25.7	23.9	22.1	0.41
6	44.2	40.2	36.3	33.6	30.4	28.7	26.9	25.3	25.9	21.4	19.9	18.4	0.60
7	37.9	34.5	31.1	28.8	26.1	24.6	23.1	21.7	22.2	18.4	17.1	15.8	0.81
8	33.1	30.2	27.2	25.2	22.8	21.5	20.2	19.0	19.5	16.1	14.9	13.8	1.06
9	29.4	26.8	24.2	22.4	20.3	19.1	18.0	16.9	17.3	14.3	13.3	12.3	1.34
10	26.5	24.1	21.8	20.1	18.2	17.2	16.2	15.2	15.6	12.9	11.9	11.0	1.66
11	24.1	22.0	19.8	18.3	16.6	15.6	14.7	13.8	14.2	11.7	10.9	10.0	2.00
12	22.1	20.1	18.2	16.8	15.2	14.3	13.5	12.6	13.0	10.7	10.0	9.2	2.38
13	20.4	18.6	16.8	15.5	14.0	13.2	12.4	11.7	12.0	9.9	9.2	8.5	2.80
14	18.9	17.2	15.6	14.4	13.0	12.3	11.5	10.8	11.1	9.2	8.5	7.9	3.24
15	17.7	16.1	14.5	13.4	12.2	11.5	10.8	10.1	10.4	8.6	8.0	7.4	3.72
16	16.6	15.1	13.6	12.6	11.4	10.8	10.1	9.5	9.7	8.0	7.5	6.9	4.24
17	15.6	14.2	12.8	11.8	10.7	10.1	9.5	8.9	9.2	...	...	...	4.78
18	14.7	13.4	12.1	11.2	10.1	9.6	9.0	8.4	8.6	...	...	...	5.36
19	13.9	12.7	11.5	10.6	...	...	...	...	...	...	...	...	5.98
20	13.3	12.1	10.9	10.1	...	...	...	...	...	...	...	...	6.62

Span, ft	Depth and weight of sections													Coefficient of deflection
	6-in			5-in			4-in				3-in			
	17¼ lb	14¾ lb	12½ lb	14¾ lb	12¼ lb	10 lb	10½ lb	9½ lb	8½ lb	7.7 lb	7½ lb	6½ lb	5.7 lb	
....	...	...	...	...	...	...	...	..	..	..	21.7	..	..	......
1	57.0	...	...	50.4	35.7	...	32.8	27.0	21.0	..	20.7	15.8	10.2	0.02
2	46.6	42.2	27.6	32.3	29.1	21.0	19.0	18.0	16.9	15.1	10.4	9.6	8.8	0.07
3	31.0	28.4	25.8	21.5	19.4	17.2	12.7	12.0	11.3	10.6	6.9	6.4	5.9	0.15
4	23.3	21.3	19.4	16.2	14.5	12.9	9.5	9.0	8.5	8.0	5.2	4.8	4.4	0.27
5	18.6	17.1	15.5	12.9	11.6	10.3	7.6	7.2	6.8	6.4	4.1	3.8	3.5	0.41
6	15.5	14.2	12.9	10.8	9.7	8.6	6.3	6.0	5.6	5.3	3.5	3.2	2.9	0.60
7	13.3	12.2	11.1	9.2	8.3	7.4	5.4	5.1	4.8	4.5	3.0	2.7	2.5	0.81
8	11.6	10.7	9.7	8.1	7.3	6.4	4.8	4.5	4.2	4.0	2.6	2.4	2.2	1.06
9	10.3	9.5	8.6	7.2	6.5	5.7	4.2	4.0	3.8	3.5	..	..	..	1.34
10	9.3	8.5	7.7	6.5	5.8	5.2	3.8	3.6	3.4	3.2	..	..	..	1.66
11	8.5	7.8	7.0	5.9	5.3	4.7	...	..	..	..	..	..	..	2.00
12	7.8	7.1	6.5	5.4	4.8	4.3	...	..	..	..	..	..	..	2.38
13	7.2	6.6	6.0	...	...	...	...	..	..	..	..	..	..	2.80
14	6.7	6.1	5.5	...	...	...	...	..	..	..	..	..	..	3.24

Loads above the upper heavy lines will cause maximum allowable shears in webs. See, also, paragraphs in text and foot-note with same, page 567, relating to web-buckling in beams

Loads below the lower broken lines will cause excessive deflections

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table V.* Safe Uniform Loads in Units of 1 000 Pounds for Steel Cha

Maximum bending stress, 16 000 lb per sq in. Beams secured against yielding si

Span, ft	Depth and weight of sections												Co cie d t
	15-in						13-in						
	55 lb	50 lb	45 lb	40 lb	35 lb	33.9 lb	50 lb	45 lb	40 lb	37 lb	35 lb	31.8 lb	
....	245.4	216.0	186.0	...	...	...	205.7	176.3	...	...	...	...	.
3	204.0	190.9	177.8	157.2	127.8	120.0	171.6	160.3	146.9	122.2	117.5	97.5	0
4	153.0	143.2	133.4	123.6	113.8	111.1	128.7	120.2	111.7	106.6	103.2	97.5	0
5	122.4	114.5	106.7	98.9	91.0	88.9	103.0	96.2	89.4	85.3	82.6	78.0	0
6	102.0	95.4	88.9	82.4	75.8	74.1	85.8	80.2	74.5	71.1	68.8	65.0	0
7	87.4	81.8	76.2	70.6	65.0	63.5	73.6	68.7	63.8	60.9	59.0	55.7	0
8	76.5	71.6	66.7	61.8	56.9	55.6	64.4	60.1	55.9	53.3	51.6	48.7	1
9	68.0	63.6	59.3	54.9	50.6	49.4	57.2	53.4	49.7	47.4	45.9	43.3	1
10	61.2	57.3	53.3	49.4	45.5	44.5	51.5	48.1	44.7	42.7	41.3	39.0	1
11	55.6	52.1	48.5	44.9	41.4	40.4	46.8	43.7	40.6	38.8	37.5	35.4	2
12	51.0	47.7	44.5	41.2	37.9	37.0	42.9	40.1	37.2	35.5	34.4	32.5	2
13	47.1	44.1	41.0	38.0	35.0	34.2	39.6	37.0	34.4	32.8	31.8	30.0	2
14	43.7	40.9	38.1	35.3	32.5	31.8	36.8	34.4	31.9	30.5	29.5	27.9	3.
15	40.8	38.2	35.6	33.0	30.3	29.6	34.3	32.1	29.8	28.4	27.5	26.0	3.
16	38.2	35.8	33.3	30.9	28.4	27.8	32.2	30.1	27.9	26.7	25.8	24.4	4.
17	36.0	33.7	31.4	29.1	26.8	26.1	30.3	28.3	26.3	25.1	24.3	22.9	4.
18	34.0	31.8	29.6	27.5	25.3	24.7	28.6	26.7	24.8	23.7	22.9	21.7	5.
19	32.2	30.1	28.1	26.0	23.9	23.4	27.1	25.3	23.5	22.4	21.7	20.5	5.
20	30.6	28.6	26.7	24.7	22.8	22.3	25.7	24.0	22.3	21.3	20.6	19.5	6.
21	29.1	27.3	25.4	23.5	21.7	21.2	24.5	22.9	21.3	20.3	19.7	18.6	7.
22	27.8	26.0	24.3	22.5	20.7	20.2	23.4	21.9	20.3	19.4	18.8	17.7	8.
23	26.6	24.9	23.2	21.5	19.8	19.3	22.4	20.9	19.4	18.5	18.0	17.0	8.
24	25.5	23.9	22.2	20.6	19.0	18.5	21.5	20.0	18.6	17.8	17.2	16.2	9.
25	24.5	22.9	21.3	19.8	18.2	17.8	20.6	19.2	17.9	17.1	16.5	15.6	10.
26	23.5	22.0	20.5	19.0	17.5	17.1	19.8	18.5	17.2	16.4	15.9	15.0	11
27	22.7	21.2	19.8	18.3	16.9	16.5	19.1	17.8	16.6	15.8	15.3	14.4	12
28	21.9	20.5	19.1	17.7	16.3	15.9	18.4	17.2	16.0	15.2	14.7	13.9	12
29	21.1	19.7	18.4	17.0	15.7	15.3	...	...	...	...	...	...	13
30	20.4	19.1	17.8	16.5	15.2	14.8	...	...	...	...	...	...	14
31	19.7	18.5	17.2	15.9	14.7	14.3	...	...	...	...	...	...	15
32	19.1	17.9	16.7	15.4	14.2	13.9	...	...	...	...	...	...	16

Loads above the upper heavy lines will cause maximum allowable shear webs. See, also, paragraphs in text and foot-note with same, page 567, rela to web-buckling in beams

Loads below the lower broken lines will cause excessive deflections

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table V * (Continued). Safe Uniform Loads in Units of 1 000 Pounds for Steel Channels

…imum bending stress, 16 000 lb per sq in. Beams secured against yielding sidewis…

Span, ft	Depth and weight of sections										Coefficient of deflection
	12-in					10-in					
	40 lb	35 lb	30 lb	25 lb	20.7 lb	35 lb	30 lb	25 lb	20 lb	15.3 lb	
..	181.9	....	....	....	....	164.6	135.2	105.8	....	....	....
2	175.1	152.6	123.1	92.6	....	123.2	110.1	97.0	76.4	49.0	0.07
3	116.7	106.2	95.8	85.3	67.2	82.1	73.4	64.7	56.0	47.6	0.15
4	87.5	79.7	71.8	64.0	56.9	61.6	55.1	48.5	42.6	35.7	0.27
5	70.0	63.7	57.5	51.2	45.5	49.3	44.0	38.8	33.6	28.5	0.41
6	58.4	53.1	47.9	42.7	38.0	41.1	36.7	32.3	28.0	23.8	0.60
7	50.0	45.5	41.1	36.6	32.5	35.2	31.5	27.7	24.0	20.4	0.81
8	43.8	39.8	35.9	32.0	28.5	30.8	27.5	24.3	21.0	17.8	1.06
9	38.9	35.4	31.9	28.4	25.3	27.4	24.5	21.6	18.7	15.9	1.34
10	35.0	31.9	28.7	25.6	22.8	24.6	22.0	19.4	16.8	14.3	1.66
11	31.8	29.0	26.1	23.3	20.7	22.4	20.0	17.6	15.3	13.0	2.00
12	29.2	26.6	23.9	21.3	19.0	20.5	18.4	16.2	14.0	11.9	2.38
13	26.9	24.5	22.1	19.7	17.5	19.0	16.9	14.9	12.9	11.0	2.80
14	25.0	22.8	20.5	18.3	16.3	17.6	15.7	13.9	12.0	10.2	3.24
15	23.3	21.2	19.2	17.1	15.2	16.4	14.7	12.9	11.2	9.5	3.72
16	21.9	19.9	18.0	16.0	14.2	15.4	13.8	12.1	10.5	8.9	4.24
17	20.6	18.7	16.9	15.1	13.4	14.5	13.0	11.4	9.9	8.4	4.78
18	19.5	17.7	16.0	14.2	12.7	13.7	12.2	10.8	9.3	7.9	5.36
19	18.4	16.8	15.1	13.5	12.0	13.0	11.6	10.2	8.8	7.5	5.98
20	17.5	15.9	14.4	12.8	11.4	12.3	11.0	9.7	8.4	7.1	6.62
21	16.7	15.2	13.7	12.2	10.8	11.7	10.5	9.2	8.0	6.8	7.30
22	15.9	14.5	13.1	11.6	10.4	11.2	10.0	8.8	7.6	6.5	8.01
23	15.2	13.9	12.5	11.1	9.9	....	....	....	....	....	8.76
24	14.6	13.3	12.0	10.7	9.5	....	....	....	....	....	9.53
25	14.0	12.8	11.5	10.2	9.1	....	....	....	....	....	10.35
26	13.5	12.3	11.1	9.8	8.8	....	....	....	....	....	11.19

Loads above the upper heavy lines will cause maximum allowable shears in webs. See, also, paragraphs in text and foot-note with same, page 567, relating to web-buckling in beams.

Loads below the lower broken lines will cause excessive deflections

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

ble V *.(Continued). Safe Uniform Loads in Units of 1 000 Pounds for Steel Channels

um bending stress, 16 000 lb per sq in. Beams secured against yielding sidew

								ections						Coeffi cient deflec tion
									7-in					
								½ lb	19¾ lb	17¼ lb	14¾ lb	12¼ lb	9.8 lb	
									89.6	71.2	50.1	41.5		......
[illegible]	72.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	50.6	46.0	41.4	36.8	[illegible]	0.07
55.9	48.0	40.2	37.4	42.5	39.0	35.5	32.0	28.7	33.7	30.7	27.6	24.6	21.4	0.15
41.9	36.0	30.1	28.0	31.8	29.2	26.6	24.0	21.5	25.3	23.0	20.7	18.4	16.1	0.27
33.5	28.8	24.1	22.4	25.5	23.4	21.3	19.2	17.2	20.2	18.4	16.6	14.7	12.9	0.41
27.9	24.0	20.1	18.7	21.2	19.5	17.7	16.0	14.4	16.9	15.3	13.8	12.3	10.7	0.60
23.9	20.6	17.2	16.0	18.2	16.7	15.2	13.7	12.3	14.4	13.1	11.8	10.5	9.2	0.81
20.9	18.0	15.1	14.0	15.9	14.6	13.3	12.0	10.8	12.6	11.5	10.4	9.2	8.0	1.06
18.6	16.0	13.4	12.5	14.2	13.0	11.8	10.7	9.6	11.2	10.2	9.2	8.2	7.1	1.34
16.8	14.4	12.1	11.2	12.7	11.7	10.6	9.6	8.6	10.1	9.2	8.3	7.4	6.4	1.66
15.2	13.1	11.0	10.2	11.6	10.6	9.7	8.7	7.8	9.2	8.4	7.5	6.7	5.8	2.00
14.0	12.0	10.1	9.3	10.6	9.7	8.9	8.0	7.2	8.4	7.7	6.9	6.1	5.4	2.38
12.9	11.1	9.3	8.6	9.8	9.0	8.2	7.4	6.6	7.8	7.1	6.4	5.7	4.9	2.80
12.0	10.3	8.6	8.0	9.1	8.4	7.6	6.9	6.2	7.2	6.6	5.9	5.3	4.6	3.24
11.2	9.6	8.0	7.5	8.5	7.8	7.1	6.4	5.7	6.7	6.1	5.5	4.9	4.3	3.72
10.5	9.0	7.5	7.0	8.0	7.3	6.7	6.0	5.4	6.3	5.7	5.2	4.6	4.0	4.24
9.9	8.5	7.1	6.6	7.5	6.9	6.3	5.6	5.1	..	..	..			4.78
9.3	8.0	6.7	6.2	7.1	6.5	5.9	5.3	4.8	..	..	..	..	..	5.36
8.8	7.6	6.3	5.9	..	..	..	..		..	..		..	..	5.98
8.4	7.2	6.0	5.6											6.6[illegible]

Depth and weight of sections													Coeffi cient deflec tion
6-in				5-in			4-in			3-in			
15½ lb	13 lb	10½ lb	8.2 lb	11½ lb	9 lb	6.7 lb	7¼ lb	6¼ lb	5.4 lb	6 lb	5 lb	4.1 lb	
				47.7			26.0			21.7	13.8	..	.....
67.4	[illegible]	[illegible]	24.0	44.4	33.0	19.0	24.4	[illegible]	14.4	14.7	13.1	[illegible]	0.03
34.7	30.8	26.9	23.1	22.2	18.9	15.8	12.2	11.1	10.1	7.4	6.6	5.8	0.07
23.2	20.5	17.9	15.4	14.8	12.6	10.5	8.1	7.4	6.7	4.9	4.4	3.9	0.15
17.4	15.4	13.4	11.6	11.1	9.5	7.9	6.1	5.6	5.1	3.7	3.3	2.9	0.27
13.9	12.3	10.8	9.2	8.9	7.6	6.3	4.9	4.5	4.1	2.9	2.6	2.3	0.41
11.6	10.3	9.0	7.7	7.4	6.3	5.3	4.1	3.7	3.4	2.5	2.2	1.9	0.60
9.9	8.8	7.7	6.6	6.3	5.4	4.5	3.5	3.2	2.9	2.1	1.9	1.7	0.81
8.7	7.7	6.7	5.8	5.5	4.7	4.0	3.0	2.8	2.5	1.8	1.6	1.5	1.06
7.7	6.8	6.0	5.1	4.9	4.2	3.5	2.7	2.5	2.2	..	..	..	1.34
6.9	6.2	5.4	4.6	4.4	3.8	3.2	2.4	2.2	2.0	..	..	..	1.66
6.3	5.6	4.9	4.2	4.0	3.4	2.9	..	.	..	..	..	..	2.00
5.8	5.1	4.5	3.9	3.7	3.2	2.6	..	.	..	..	..	..	2.38
5.3	4.7	4.1	3.6	...	...	..	..	..	..	..	..	..	2.80
5.0	4.4	3.8	3.3				.		.	..	..	..	3.24

ds above the upper heavy lines will cause maximum allowable shears i
See, also, paragraphs in text and foot-note with same, page 567, relatin
>-buckling in beams
ds below the lower broken lines will cause excessive deflections

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table VI.* Safe Uniform Loads in Units of 1 000 Pounds for Steel H Beams

Maximum bending stress, 16 000 lb per sq in. Secured against yielding sidewise

Span, ft	Depth and weight of sections				Coefficients of deflection
	8-in 34.3-lb	6-in 24.1-lb	5-in 18.9-lb	4-in 13.8-lb	
... ...	....	....	....	25.0	....
3	....	....	31.3	19.0	0.15
4	....	37.6	25.4	14.3	0.27
5	....	32.1	20.3	11.4	0.41
....	60.0	....	....	....	....
6	51.3	26.7	16.9	9.5	0.60
7	44.0	22.9	14.5	8.1	0.81
8	38.5	20.1	12.7	7.1	1.06
9	34.2	17.8	11.3	6.3	1.34
10	30.8	16.0	10.1	5.7	1.66
11	28.0	14.6	9.2	....	2.00
12	25.6	13.4	8.5	....	2.38
13	23.7	12.3	....	....	2.80
14	22.0	11.5	....	....	3.24
15	20.5	....	....	....	3.72
16	19.2	....	....	....	4.24
17	18.1	....	....	....	4.78
18	17.1	....	....	....	5.36

Table VII.† Safe Uniform Loads in Pounds for Small Steel Channels, or Grooved Steel

Computed for a fiber-stress of 16 000 lb per sq in

Secured against yielding sidewise

Section-number	Depth, in	Weight per foot, lb	Span in feet							
			2	2.5	3	3.5	4	4.5	5	6
1	2¼	3.80	3 785	3 028	2 523	2 163	1 892	1 682	1 514	1 26[illegible]
2	2	2.90	2 560	2 048	1 706	1 463	1 280	1 138	1 024	85[illegible]
3	2	3.60	2 880	2 304	1 920	1 643	1 440	1 280	1 152	96[illegible]
4	2	3.60	3 120	2 496	2 080	1 783	1 560	1 386	1 248	1 04[illegible]
5	2	2.60	2 256	1 804	1 504	1 289	1 128	1 000	902	75[illegible]
6	2	2.00	1 418	1 134	945	810	709	630	567	47[illegible]
7	1¾	1.13	907	726	605	518	454	403	363	30[illegible]
8	1½	1.32	768	614	512	439	384	341	307	25[illegible]
9	1½	1.46	868	694	578	496	434	386	347	28[illegible]
10	1¼	0.94	475	380	316	271	237	211	190	....
11	1⅛	1.12	469	375	313	268	234	208	188	....
12	1⅛	1.00	437	350	291	250	218	194	175	....
13	1	0.83	336	268	224	192	168	....	....	....
14	1	0.68	266	212	177	152	133	....	....	....
15	⅞	0.67	224	180	149	128	112	....	....	....
16	⅞	0.69	229	183	152	130	....	....	....	....
17	¾	0.53	133	106	88	....	....	....	....	....

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

† Compiled by F. E. Kidder. See note on page 570.

Table VIII.* Safe Uniform Loads in Units of 1 000 Pounds for Steel Angles with Equal Legs. (See page 566.)

Neutral Axis Parallel to Either Leg

Maximum bending stress, 16 000 lb per sq in. Secured against yielding sidewise

in	in	Safe load	Maximum span, 360 × deflection: Safe load	Length, ft
		186.99	8.31	22.5
		177 81	7.87	22.6
		168 53	7 43	22 7
		159 15	6 98	22 8
		149 55	6 53	22 9
		139 84	6 08	23 0
		130 03	5 63	23 1
		120 00	5 18	23 2
		109 87	4.73	23 2
		99 63	4 28	23 3
		89 28	3.82	23.4
		91 41	5 48	16 7
		86 51	5 16	16 8
		81 39	4 84	16 8
		76.27	4 51	16 9
		71 04	4 18	17 0
		65.81	3.85	17.1
6×6	5/8	60 37	3 51	17 2
6×6	9/16	54 83	3 17	17 3
6×6	1/2	49 17	2 83	17 4
6×6	7/16	43 41	2.48	17.5
6×6	3/8	37 65	2 14	17 6
5×5	1	61 87	4 55	13.6
5×5	15/16	58 56	4 28	13.7
5×5	7/8	55 15	4 00	13.8
5×5	13/16	51.73	3.73	13.9
5×5	3/4	48.32	3 45	14.0
5×5	11/16	44.80	3.18	14.1
5×5	5/8	41.17	2 90	14.2
5×5	9/16	37.44	2.62	14 3
5×5	1/2	33.60	2.34	14.4
5×5	7/16	29.76	2 06	14 5
5×5	3/8	25 81	1 78	14.5
4×4	13/16	32 11	2 95	10.9
4×4	3/4	29 97	2 73	11.0
4×4	11/16	27 84	2 51	11.1
4×4	5/8	25 60	2 29	11 2
4×4	9/16	23.36	2 07	11.3
4×4	1/2	21.01	1.85	11.4
4×4	7/16	18 67	1 63	11.4
4×4	3/8	16 21	1.41	11.5
4×4	5/16	13 76	1 19	11.6
4×4	1/4	11.20	0.96	11.7

Size, in	Thickness, in	1-ft span: Safe load	Maximum span, 360 × deflection: Safe load	Length, ft
3½×3½	13/16	24 00	2 55	9 4
3½×3½	3/4	22.51	2 37	9.5
3½×3½	11/16	20 91	2 18	9.6
3½×3½	5/8	19.31	2 00	9.7
3½×3½	9/16	17 60	1 81	9.7
3½×3½	1/2	15 89	1 62	9.8
3½×3½	7/16	14 08	1 42	9 9
3½×3½	3/8	12.27	1.23	10.0
3½×3½	5/16	10.45	1 04	10.1
3½×3½	1/4	8 43	0 83	10.2
3 ×3	5/8	13 87	1.69	8.2
3 ×3	9/16	12 69	1 53	8 3
3 ×3	1/2	11.41	1.37	8.3
3 ×3	7/16	10 13	1 21	8.4
3 ×3	3/8	8 85	1 04	8.5
3 ×3	5/16	7 57	0.88	8.6
3 ×3	1/4	6.19	0.71	8 7
2½×2½	1/2	7.79	1.15	6.8
2½×2½	7/16	6 93	1 01	6 9
2½×2½	3/8	6.08	0 87	7.0
2½×2½	5/16	5 12	0.72	7 1
2½×2½	1/4	4 16	0.58	7.2
2½×2½	3/16	3 20	0.44	7.3
2½×2½	1/8	2 13	0.29	7.4
2 ×2	7/16	4 27	0 79	5.4
2 ×2	3/8	3.73	0.68	5.5
2 ×2	5/16	3 20	0 57	5.6
2 ×2	1/4	2 67	0.46	5.7
2 ×2	3/16	2 03	0 35	5.8
2 ×2	1/8	1 39	0 24	5.8
1¾×1¾	7/16	3.20	0 68	4.7
1¾×1¾	3/8	2.77	0 60	4.7
1¾×1¾	5/16	2.45	0.51	4.8
1¾×1¾	1/4	2 03	0 41	4.9
1¾×1¾	3/16	1 49	0 30	5.0
1¾×1¾	1/8	1.07	0 21	5.1
1½×1½	3/8	2 03	0 51	4.0
1½×1½	5/16	1.71	0 42	4.1
1½×1½	1/4	1.39	0 33	4.2
1½×1½	3/16	1.07	0.25	4.3
1½×1½	1/8	0.77	0.17	4.4
1¼×1¼	5/16	1 17	0.36	3.3
1¼×1¼	1/4	0.97	0.29	3.4
1¼×1¼	3/16	0.76	0.22	3.5
1¼×1¼	1/8	0.52	0.14	3.6
1 ×1	1/4	0.60	0.28	2.6
1 ×1	3/16	0.47	0.17	2.7
1 ×1	1/8	0.33	0.12	2.8

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table IX.* Safe Uniform Loads in Units of 1 000 Pounds of Steel Angles with Unequal Legs. (See page 566.)

Neutral Axis Parallel to Shorter Leg

nding stress, 16 000 lb per sq in. Secured against yielding sidewise

Size, in	Thickness, in	1-ft span	Maximum span, 360 × deflection		Size, in	Thickness, in	1-ft span	Maximum span, 360 × deflection	
		Safe load	Safe load	Length, ft			Safe load	Safe load	Length, ft
	1	161.17	7.49	21.5	6×3½	1	83.52	5.57	15.0
	15/16	152.21	7.04	21.6	6×3½	15/16	79.04	5.24	15.1
	7/8	143.04	6.59	21.7	6×3½	7/8	74.45	4.90	15.2
	13/16	133.87	6.14	21.8	6×3½	13/16	69.87	4.57	15.3
	3/4	124.48	5.68	21.9	6×3½	3/4	65.07	4.23	15.4
	11/16	114.88	5.22	22.0	6×3½	11/16	60.27	3.89	15.5
	5/8	105.28	4.76	22.1	6×3½	5/8	55.36	3.55	15.6
	9/16	95.47	4.30	22.2	6×3½	9/16	50.35	3.21	15.7
	1/2	85.55	3.84	22.3	6×3½	1/2	45.23	2.86	15.8
	7/16	75.41	3.37	22.4	6×3½	7/16	40.00	2.52	15.9
					6×3½	3/8	34.67	2.17	16.0
[illegible]	1	146.03	7.53	19.4	6×3½	5/16	29.23	1.83	16.0
X3½	15/16	138.03	7.08	19.5					
X3½	7/8	129.92	6.63	19.6					
X3½	13/16	121.60	6.17	19.7	5×4	7/8	53.23	4.00	13.3
X3½	3/4	113.17	5.72	19.8	5×4	13/16	50.03	3.73	13.4
X3½	11/16	104.58	5.23	19.9	5×4	3/4	[illegible] 61	3.46	13.5
X3½	5/8	95.79	4.78	20.0	5×4	11/16	43.20	3.19	13.5
X3½	9/16	86.93	4.32	20.1	5×4	5/8	39.79	2.92	13.6
X3½	1/2	77.97	3.86	20.2	5×4	9/16	36.16	2 [illegible]	13.7
X3½	7/16	68.80	3.39	20.3	5×4	1/2	32.53	2.36	13.8
					5×4	7/16	28.80	2.07	13.9
7X3½	1	112.85	6.52	17.3	5×4	3/8	24.96	1.78	14.0
7X3½	15/16	106.67	6.13	17.4					
7X3½	7/8	100.48	5.75	17.5					
X3½	13/16	94.08	5.36	17.6	5×3½	7/8	52.05	4.04	12.9
X3½	3/4	87.68	4.97	17.6	5×3½	13/16	48.85	3.76	13.0
X3½	11/16	81.07	4.58	17.7	5×3½	3/4	[illegible] 65	3.49	13.1
X3½	5/8	74.35	4.18	17.8	5×3½	11/16	42.35	3.21	13.2
X3½	9/16	67.52	3.77	17.9	5×3½	5/8	38.93	2.93	13.3
X3½	1/2	60.59	3.37	18.0	5×3½	9/16	35.41	2.64	13.4
X3½	7/16	53.44	2.96	18.1	5×3½	1/2	31.89	2.36	13.5
X3½	3/8	46.19	2.54	18.2	5×3½	7/16	28.16	2.07	13.6
					5×3½	3/8	24.43	1.79	13.7
X4	1	85.55	5.56	15.4	5×3½	5/16	20.69	1.51	13.7
X4	15/16	80.96	5.22	15.5					
X4	7/8	76.27	4.89	15.6	5×3	13/16	47.47	3.77	12.6
X4	13/16	71.47	4.55	15.7	5×3	3/4	44.37	3.49	12.7
X4	3/4	66.67	4.22	15.8	5×3	11/16	41.17	3.22	12.8
X4	11/16	61.65	3.88	15.9	5×3	5/8	37.87	2.94	12.9
X4	5/8	56.64	3.54	16.0	5×3	9/16	34.45	2.65	13.0
X4	9/16	51.52	3.20	16.1	5×3	1/2	31.04	2.37	13.1
X4	1/2	46.19	2.85	16.2	5×3	7/16	27.52	2.09	13.2
X4	7/16	40.85	2.51	16.3	5×3	3/8	23.89	1.80	13.3
X4	3/8	35.41	2.16	16.4	5×3	5/16	20.16	1.51	13.4

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table IX * (Continued). Safe Uniform Loads in Units of 1 000 Pounds for Steel Angles with Unequal Legs. (See page 566.)

Neutral Axis Parallel to Shorter Leg

Maximum bending stress, 16 000 lb per sq in. Secured against yielding sidewi

Size, in	Thickness, in	1-ft span	Maximum span, 360 × deflection	
		Safe load	Safe load	Length, ft
4½×3	13/16	38.61	3.36	11.5
4½×3	3/4	36.05	3.11	11.6
4½×3	11/16	33.49	2.87	11.7
4½×3	5/8	30.83	2.62	11.8
4½×3	9/16	28.16	2.38	11.8
4½×3	1/2	25.28	2.13	11.9
4½×3	7/16	22.40	1.87	12.0
4½×3	3/8	19.52	1.61	12.1
4½×3	5/16	16.43	1.35	12.2
4 ×3½	13/16	31.15	2.94	10.6
4 ×3½	3/4	29.23	2.73	10.7
4 ×3½	11/16	27.20	2.52	10.8
4 ×3½	5/8	25.07	2.30	10.9
4 ×3½	9/16	22.93	2.08	11.0
4 ×3½	1/2	20.69	1.86	11.1
4 ×3½	7/16	18.35	1.64	11.2
4 ×3½	3/8	16.00	1.41	11.3
4 ×3½	5/16	13.44	1.18	11.4
4 ×3	13/16	30.61	2.97	10.3
4 ×3	3/4	28.59	2.75	10.4
4 ×3	11/16	26.56	2.53	10.5
4 ×3	5/8	24.53	2.31	10.6
4 ×3	9/16	22.40	2.09	10.7
4 ×3	1/2	20.16	1.87	10.8
4 ×3	7/16	17.92	1.64	10.9
4 ×3	3/8	15.57	1.42	11.0
4 ×3	5/16	13.12	1.19	11.0
4 ×3	1/4	10.67	0.96	11.1
3½×3	13/16	23.47	2.57	9.1
3½×3	3/4	21.87	2.38	9.2
3½×3	11/16	20.37	2.19	9.3
3½×3	5/8	18.77	2.00	9.4
3½×3	9/16	17.17	1.81	9.5
3½×3	1/2	15.47	1.62	9.5
3½×3	7/16	13.76	1.43	9.6
3½×3	3/8	12.05	1.24	9.7
3½×3	5/16	10.24	1.05	9.8
3½×3	1/4	8.32	0.84	9.9
3½×2½	11/16	19.73	2.19	9.0
3½×2½	5/8	18.24	2.00	9.1
3½×2½	9/16	16.64	1.82	9.1
3½×2½	1/2	15.04	1.63	9.2
3½×2½	7/16	13.44	1.44	9.3
3½×2½	3/8	11.73	1.24	9.4
3½×2½	5/16	9.92	1.04	9.5
3½×2½	1/4	8.00	0.83	9.6

Size, in	Thickness, in	1-ft span	Maximum span, 36[0] × deflect[ion]
		Safe load	Safe load
3 ×2½	9/16	12.27	1.53
3 ×2½	1/2	11.09	1.37
3 ×2½	7/16	9.92	1.22
3 ×2½	3/8	8.64	1.06
3 ×2½	5/16	7.36	0.89
3 ×2½	1/4	5.97	0.71
3 ×2	1/2	10.67	1.39
3 ×2	7/16	9.49	1.22
3 ×2	3/8	8.32	1.05
3 ×2	5/16	7.04	0.88
3 ×2	1/4	5.76	0.71
2½×2	1/2	7.47	1.15
2½×2	7/16	6.72	1.02
2½×2	3/8	5.87	0.88
2½×2	5/16	5.01	0.74
2½×2	1/4	4.05	0.59
2½×2	3/16	3.09	0.44
2½×2	1/8	2.13	0.30
2½×1½	5/16	4.69	0.73
2½×1½	1/4	3.84	0.59
2½×1½	3/16	2.99	0.45
2¼×1½	1/2	5.76	1.02
2¼×1½	7/16	5.12	0.90
2¼×1½	3/8	4.48	0.77
2¼×1½	5/16	3.84	0.65
2¼×1½	1/4	3.20	0.53
2¼×1½	3/16	2.45	0.40
2 ×1½	3/8	3.63	0.70
2 ×1½	5/16	3.09	0.58
2 ×1½	1/4	2.56	0.47
2 ×1½	3/16	1.92	0.35
2 ×1½	1/8	1.39	0.24
2 ×1¼	1/4	2.45	0.47
2 ×1¼	3/16	1.92	0.36
1¾×1¼	1/4	1.92	0.42
1¾×1¼	3/16	1.49	0.32
1¾×1¼	1/8	1.00	0.21
1½×1¼	5/16	1.71	0.44
1½×1¼	1/4	1.39	0.35
1½×1¼	3/16	1.07	0.26

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table X.* Safe Uniform Loads in Units of 1 000 Pounds for Steel Angles with Unequal Legs. (See page 564.)

Neutral Axis Parallel to Longer Leg

Maximum bending stress, 16 000 lb per sq in. Secured against yielding sidewise

Size, in	Thickness, in	1-ft span: Safe load	Maximum span, 360 × deflection: Safe load	Maximum span, 360 × deflection: Length, ft
	1	95.15	5.44	17.5
	15/16	89.92	5.11	17.6
	7/8	84.69	4.79	17.7
	13/16	79.36	4.45	17.8
	3/4	73.92	4.13	17.9
	11/16	68.37	3.80	18.0
	5/8	62.72	3.48	18.0
	9/16	56.96	3.15	18.1
…×6	1/2	51.09	2.81	18.2
…×6	7/16	45.12	2.47	18.3
…×3½	1	32.21	3.10	10.4
…×3½	15/16	30.40	2.90	10.5
…×3½	7/8	28.69	2.71	10.6
…×3½	13/16	26.88	2.52	10.7
…×3½	3/4	25.07	2.33	10.8
…×3½	11/16	23.15	2.13	10.9
…×3½	5/8	21.33	1.94	11.0
…×3½	9/16	19.41	1.74	11.1
…×3½	1/2	17.49	1.57	11.2
…×3½	7/16	15.57	1.38	11.3
…×3½	1	31.57	3.10	10.2
…×3½	15/16	29.87	2.90	10.3
…×3½	7/8	28.16	2.71	10.4
…×3½	13/16	26.45	2.52	10.5
…×3½	3/4	24.64	2.33	10.6
…×3½	11/16	22.83	2.14	10.7
…×3½	5/8	21.01	1.95	10.8
…×3½	9/16	19.20	1.76	10.9
…×3½	1/2	17.28	1.57	11.0
…×3½	7/16	15.36	1.38	11.1
…×3½	3/8	13.44	1.19	11.2
…4	1	40.43	3.55	11.4
…4	15/16	38.29	3.33	11.5
…4	7/8	36.16	3.12	11.6
…4	13/16	33.92	2.90	11.7
…4	3/4	31.68	2.69	11.8
…4	11/16	29.44	2.47	11.9
…4	5/8	27.09	2.26	12.0
…4	9/16	24.64	2.05	12.0
…4	1/2	22.19	1.84	12.1
…4	7/16	19.73	1.62	12.2
…4	3/8	17.07	1.39	12.3

Size, in	Thickness, in	1-ft span: Safe load	Maximum span, 360 × deflection: Safe load	Maximum span, 360 × deflection: Length, ft
6×3½	1			
6×3½	15/16			
6×3½	7/8			
6×3½	13/16			
6×3½	3/4			
6×3½	11/16			
6×3½	5/8			
6×3½	9/16			
6×3½	1/2			
6×3½	7/16			
6×3½	3/8			
6×3½	5/16			
5×4	7/8			
5×4	13/16			
5×4	3/4			
5×4	11/16			
5×4	5/8			
5×4	9/16			
5×4	1/2			
5×4	7/16			
5×4	3/8			
5×3½	7/8			
5×3½	13/16			
5×3½	3/4			
5×3½	11/16			
5×3½	5/8			
5×3½	9/16			
5×3½	1/2			
5×3½	7/16			
5×3½	3/8			
5×3½	5/16			
5×3	13/16			
5×3	3/4			
5×3	11/16			
5×3	5/8			
5×3	9/16			
5×3	1/2			
5×3	7/16			
5×3	3/8			
5×3	5/16			

* From *Pocket Companion*, Carnegie Steel Company, Pittsburgh, Pa.

Table X * (Continued). Safe Uniform Loads in Units of 1 000 Pounds f[illegible] Steel Angles with Unequal Legs. (See page 566.)

Neutral Axis Parallel to Longer Leg

Maximum bending stress, 16 000 lb per sq in. Secured against yielding sidewis[illegible]

Size, in	Thickness, in	1-ft span	Maximum span, 360 × deflection		Size, in	Thickness, in	1-ft span	Maximum span, 360 × deflection	
		Safe load	Safe load	Length, ft			Safe load	Safe load	Length, ft
4½×3	13/16	18.24	2.15	8.5	3 ×2½	9/16	8.75	1.25	[illegible]
4½×3	3/4	17.07	1.99	8.6	3 ×2½	1/2	7.89	1.12	[illegible]
4½×3	11/16	15.89	1.83	8.7	3 ×2½	7/16	7.04	0.99	[illegible]
4½×3	5/8	14.61	1.67	8.8	3 ×2½	3/8	6.19	0.85	[illegible]
4½×3	9/16	13.33	1.51	8.8	3 ×2½	5/16	5.23	0.72	[illegible]
4½×3	1/2	12.05	1.35	8.9	3 ×2½	1/4	4.27	0.58	[illegible]
4½×3	7/16	10.77	1.19	9.0					
4½×3	3/8	9.39	1.03	9.1	3 ×2	1/2	5.01	0.88	[illegible]
4½×3	5/16	8.00	0.87	9.2	3 ×2	7/16	4.48	0.77	[illegible]
4 ×3½	13/16	24.53	2.56	9.6	3 ×2	3/8	3.95	0.67	[illegible]
4 ×3½	3/4	22.93	2.37	9.7	3 ×2	5/16	3.41	0.57	[illegible]
4 ×3½	11/16	21.33	2.18	9.8	3 ×2	1/4	2.77	0.46	[illegible]
4 ×3½	5/8	19.63	1.98	9.9					
4 ×3½	9/16	17.92	1.79	10.0	2½×2	1/2	4.91	0.89	[illegible]
4 ×3½	1/2	16.21	1.60	10.1	2½×2	7/16	4.37	0.78	[illegible]
4 ×3½	7/16	14.40	1.41	10.2	2½×2	3/8	3.84	0.67	[illegible]
4 ×3½	3/8	12.59	1.22	10.3	2½×2	5/16	3.31	0.57	[illegible]
4 ×3½	5/16	10.67	1.03	10.4	2½×2	1/4	2.67	0.46	[illegible]
4 ×3	13/16	17.92	2.15	8.3	2½×2	3/16	2.13	0.35	[illegible]
4 ×3	3/4	16.75	1.99	8.4	2½×2	1/8	1.49	0.23	[illegible]
4 ×3	11/16	15.57	1.83	8.5					
4 ×3	5/8	14.40	1.67	8.6	2½×1½	5/16	1.81	0.41	[illegible]
4 ×3	9/16	13.12	1.51	8.7	2½×1½	1/4	1.49	0.33	[illegible]
4 ×3	1/2	11.84	1.35	8.8	2½×1½	3/16	1.17	0.25	[illegible]
4 ×3	7/16	10.56	1.19	8.9					
4 ×3	3/8	9.28	1.03	8.9	2¼×1½	1/2	2.77	0.67	[illegible]
4 ×3	5/16	7.89	0.87	9.0	2¼×1½	7/16	2.45	0.58	[illegible]
4 ×3	1/4	6.40	0.70	9.1	2¼×1½	3/8	2.13	0.50	[illegible]
3½×3	13/16	17.60	2.17	8.1	2¼×1½	5/16	1.81	0.41	[illegible]
3½×3	3/4	16.43	2.01	8.2	2¼×1½	1/4	1.49	0.33	[illegible]
3½×3	11/16	15.36	1.85	8.3	2¼×1½	3/16	1.17	0.25	[illegible]
3½×3	5/8	14.19	1.69	8.4					
3½×3	9/16	12.91	1.52	8.5	2 ×1½	3/8	2.13	0.51	[illegible]
3½×3	1/2	11.73	1.36	8.6	2 ×1½	5/16	1.81	0.42	[illegible]
3½×3	7/16	10.45	1.20	8.7	2 ×1½	1/4	1.49	0.34	[illegible]
3½×3	3/8	9.07	1.04	8.7	2 ×1½	3/16	1.17	0.26	[illegible]
3½×3	5/16	7.68	0.87	8.8	2 ×1½	1/8	0.80	0.17	[illegible]
3½×3	1/4	6.19	0.70	8.9	2 ×1¼	1/4	1.04	0.28	[illegible]
3½×2½	11/16	10.56	1.51	7.0	2 ×1¼	3/16	0.80	0.21	[illegible]
3½×2½	5/8	9.81	1.39	7.1	1¾×1¼	1/4	1.01	0.28	[illegible]
3½×2½	9/16	8.96	1.26	7.1	1¾×1¼	3/16	0.80	0.22	[illegible]
3½×2½	1/2	8.11	1.13	7.2	1¾×1¼	1/8	0.56	0.15	[illegible]
3½×2½	7/16	7.25	0.99	7.3					
3½×2½	3/8	6.29	0.85	7.4	1½×1¼	5/16	1.17	0.34	[illegible]
3½×2½	5/16	5.33	0.71	7.5	1½×1¼	1/4	0.99	0.28	[illegible]
3½×2½	1/4	4.37	0.58	7.6	1½×1¼	3/16	0.78	0.22	[illegible]

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

ble XI.* Safe Uniform Loads in Units of 1 000 Pounds for Steel Tee
Neutral Axis Parallel to Flange. (See page 566.)

Maximum bending stress, 16 000 lb per sq in. Secured against yielding side

EQUAL FLANGE AND STEM

Size: Flnge in	Size: Stem in	Weight per foot, lb	1-ft span: Safe load	Maximum span, 360 × deflection: Safe load	Maximum span, 360 × deflection: L'gth ft	Size: Flnge in	Size: Stem in	Weight per foot, lb	1-ft span: Safe load	Maximum span, 360 × deflection: Safe load
6½	6½	19.8	52.80	2.77	19.1	2¼	2¼	4.9	4.37	0.69
4	4	13.5	21.55	1.89	11.4	2¼	2¼	4.1	3.41	0.53
4	4	10.5	16.85	1.45	11.6	2	2	4.3	3.31	0.59
3½	3½	11.7	16.32	1.65	9.9	2	2	3.56	2.77	0.49
3½	3½	9.2	12.69	1.27	10.0	1¾	1¾	3.09	2.03	0.41
3	3	9.9	11.73	1.41	8.3	1½	1½	2.47	1.49	0.36
3	3	8.9	10.45	1.24	8.4	1½	1½	1.94	1.17	0.27
3	3	7.8	9.17	1.08	8.5	1¼	1¼	2.02	1.01	0.30
3	3	6.7	7.89	0.92	8.6	1¼	1¼	1.59	0.78	0.22
2½	2½	6.4	6.29	0.90	7.0	1	1	1.25	0.49	0.18
2½	2½	5.5	5.33	0.75	7.1	1	1	0.89	0.35	0.12
2¼	2¼	4.9	4.37	0.69	6.3	....	....	......	....	...

UNEQUAL FLANGE AND STEM

Size: Flnge in	Size: Stem in	Weight per foot, lb	1-ft span: Safe load	Maximum span, 360 × deflection: Safe load	Maximum span, 360 × deflection: L'gth ft	Size: Flnge in	Size: Stem in	Weight per foot, lb	1-ft span: Safe load	Maximum span, 360 × deflection: Safe load
5	3	13.4	11.41	1.25	9.1	3½	3	10.8	12.05	1.42
5	2½	10.9	8.96	1.20	7.5	3½	3	8.5	9.49	1.09
4½	3½	15.7	22.72	2.37	9.6	3½	3	7.5	9.07	1.04
4½	3	9.8	9.71	1.07	9.1	3	4	11.7	20.69	1.92
4½	3	8.4	8.32	0.90	9.2	3	4	10.5	18.35	1.68
4½	2½	9.2	6.72	0.87	7.7	3	4	9.2	16.11	1.47
4½	2½	7.8	5.76	0.74	7.8	3	3½	10.8	15.89	1.66
4	5	15.3	33.39	2.40	13.9	3	3½	9.7	14.19	1.46
4	5	11.9	25.92	1.84	14.1	3	3½	8.5	12.37	1.26
4	4½	14.4	27.09	2.13	12.6	3	2½	7.1	6.40	0.89
4	4½	11.2	21.12	1.65	12.8	3	2½	6.1	5.55	0.76
4	3	9.2	9.60	1.08	8.9	2½	3	7.1	8.96	1.08
4	3	7.8	8.21	0.90	9.1	2½	3	6.1	7.68	0.91
4	2½	8.5	6.61	0.87	7.6	2½	1¼	2.87	0.93	0.25
4	2½	7.2	5.65	0.73	7.7	2	1½	3.09	1.60	0.36
4	2	7.8	4.27	0.70	6.1	1½	2	2.45	2.03	0.37
4	2	6.7	3.63	0.59	6.2	1½	1¼	1.25	0.57	0.15
3½	4	12.6	21.12	1.90	11.1	1¼	⅝	0.88	0.14	0.07
3½	4	9.8	16.53	1.46	11.3	....	....	......	....	...

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Bethlehem I Beams.* BETHLEHEM I BEAMS from 8 to 24 in in depth clusive, have the same strength, or section-modulus, as STANDARD BEAMS of same depth. Bethlehem beams, due to the proportions of the sections, w generally 10% less than standard beams of the same depth and strength. example (Table VI, page 357), a Bethlehem 15-in I beam, weighing 54 lb ft, has a section-modulus of 81.3. The corresponding standard section (T IV, page 354) is a 15-in I beam weighing 60.8 lb per ft, with a section-mod of 81.2. Therefore, for equal strength, the Bethlehem beam weighs 6.8 lb p less than the standard beam, or saving over 10% in weight. Similar c parisons with other sizes of the standard beams previously rolled by the r of this country show that the Bethlehem I beams afford an equal carr capacity, but with practically 10% less weight of metal.

Thickness of Webs and Flanges. It is claimed that the WEBS of stan beams are much thicker than required for a scientifically proportioned sec It is impossible to reduce the WEB-THICKNESS in the ordinary mill, but the Grey Mill webs of the desired thickness can be produced. By addin the FLANGES part of the metal thus saved, the strength of the beam is m tained, thereby affording a lighter section of the same strength. The v FLANGES give increased lateral stiffness, which commends the use of such b in many cases, where the NARROW FLANGES and lack of sufficient lateral rig prevent the use of ordinary standard beams.

Depth and Weight of Bethlehem Beams. Formerly the heaviest b rolled in this country were 24 in deep, weighed 115 lb per ft, and had a sec modulus of 246.3. Whenever greater strength was required, a riveted g was necessary. Bethlehem beams are rolled to a maximum depth of 3 weigh 200 lb per ft, and have a section-modulus of 610, or two and one times the strength of the largest beam previously rolled. The opportunit using ROLLED BEAMS instead of BUILT-UP RIVETED GIRDERS is, therefore, gr increased. These rolled beams and girders afford a saving in WEIGHT OF M and also a large economy in COST OF FABRICATION, as they do not requir punching, assembling and riveting necessary for building a riveted girder

Bethlehem Girder Beams.* BETHLEHEM GIRDER BEAMS, from 8 to in depth, inclusive, have a strength, or section-modulus, equal to that of minimum-weight STANDARD I BEAMS of the same depth. The girder b however, weighs generally 12½% less than the combined weight of the standard beams, not considering the saving in weight of separators neede assembling the standard beams into a girder. For example, a Bethlehem girder beam, weighing 73 lb per ft has a section-modulus of 117.8 (Table page 358). Two standard 15-in I beams, each weighing 42 lb per ft, together a like section-modulus of 117.8 (Table IV, page 354). Thus, for depth and strength, the girder beam weighs 11 lb per ft less than the standard beams. This is a saving of 13% in weight, not including separa which would add at least 2½ lb per ft more to the weight of the assem girder. In this case a total saving of 16% in weight is afforded by the Be hem girder beam, besides the saving in the cost of assembling the stan beams into a girder.

Safe Uniformly Distributed Loads for Bethlehem I Beams and Gi Beams. Tables XII * and XIII,* pages 594 to 602, give the SAFE UNIFO DISTRIBUTED LOADS in tons of 2 000 lb, on Bethlehem girder beams and I b for a maximum fiber-stress of 16 000 lb per sq in. The tabular loads in

* Adapted by permission from the Catalogue of Bethlehem Structural Shapes, B hem Steel Company, South Bethlehem, Pa.

e weight of the beam, which must be deducted to obtain the net load a beam ill support. Safe loads for INTERMEDIATE OR HEAVIER WEIGHTS of beams can obtained from the separate COLUMN OF CORRECTIONS, given for each size his last column of the table states the increase in safe load for each pound increase in weight per foot of beam. If the load is CONCENTRATED AT THE DDLE OF THE SPAN, the safe load is one-half the safe uniformly distributed d for the same span. The SAFE LOADS ON SHORT SPANS may be limited by e shearing strength of the web, instead of by the maximum fiber-stress lowed in the flanges. This limit is indicated in the tables by the heavy hori- ntal lines. The loads given above these lines are greater than the SAFE CRIP- LING OF BUCKLING STRENGTH OF THE WEB, and must not be used unless the ebs are stiffened.* In such cases it will generally be advisable to select a avier beam with a thicker web. To use these tables for other spans, or for her distribution of the loading, see explanation, page 566. To use these tables r beams YIELDING LATERALLY, see Lateral Deflection, pages 566 and 670.

blique Loading of Angles Used as Beams †

Oblique Loading of Purlins on oping Roofs. (See, also, pages 3, 1169 and 1170.) The preceding bles VIII, IX and X for safe loads angles are based on the neutral is being parallel to one of the legs. hen this is not the case, as in roof- rlins (Fig. 10), the strength of a an angle may be found by taking section-modulus from Table XI A d using the fundamental formula r flexure (page 557). It should be ted that purlins set as at (*a*) are onger than (*b*), Fig. 9A.

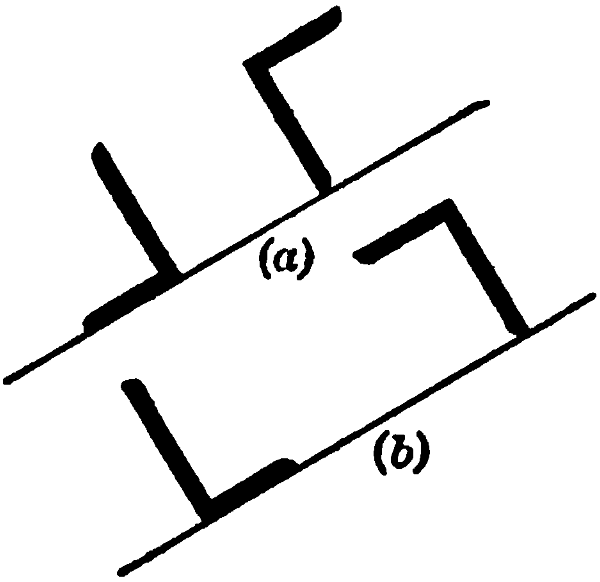


Fig. 9A. Strong and Weak Setting of Angle-Purlins on Sloping Roofs

ble XI A. Section-Moduli of Angle-Purlins Set at Right-Angles to Rafters, as in (a) Fig. 9A, and Free to Move in Any Direction. Loading Vertical.

Purlin	Slope of roof in inches per foot						
	0	1	2	4	5	6	8
× 2 × 1/4 angle	0.18	0.19	0.20	0.22	0.24	0.26	0.31
½ × 2 × 1/4 angle	0.30	0.31	0.33	0.38	0.41	0.44	0.49
½ × 2½ × 1/4 angle	0.31	0.32	0.33	0.37	0.39	0.42	0.48
½ × 2½ × 3/8 angle	0.42	0.44	0.46	0.52	0.55	0.60	0.68
× 2½ × 1/4 angle	0.44	0.46	0.49	0.56	0.60	0.65	0.74
× 2½ × 3/8 angle	0.64	0.67	0.71	0.82	0.89	0.95	1.06
½ × 2½ × 1/4 angle	0.56	0.59	0.64	0.76	0.83	0.89	0.84
½ × 2½ × 3/8 angle	0.84	0.89	0.96	1.14	1.22	1.29	1.19
× 3 × 1/4 angle	0.75	0.80	0.86	1.02	1.11	1.18	1.27
× 3 × 3/8 angle	1.11	1.18	1.26	1.47	1.58	1.70	1.76
× 3½ × 5/16 angle	1.48	1.55	1.66	1.96	2.12	2.30	2.06
× 3½ × 3/8 angle	1.76	1.86	1.99	2.34	2.52	2.71	2.42
× 4 × 3/8 angle	2.50	2.66	2.87	3.41	3.70	4.00	3.17
× 4 × 1/2 angle	3.30	3.52	3.79	4.52	4.84	5.18	4.09

* See paragraphs and foot-note, page 567, relating to web-buckling of beams.
† From Notes by Robins Fleming.

Table XII. Safe Uniform Loads in Tons of 2 000 Pounds for Bethlehem Girder Beams

Beams secured against yielding sidewise

Span, ft	30-in G		Add for each lb increase in weight	28-in G		Add for each lb increase in weight	26-in G		Add f each incre in weig
	G30 a	G30		G28 a	G28		G26 a	G26	
	200 lb	180 lb		180 lb	165 lb		160 lb	150 lb	
18	180.75	161.87	0.44	153.75	138.89	0.41	128.11	117.47	0.3
19	171.24	153.35	0.41	145.66	131.58	0.39	121.37	111.29	0.3
20	162.68	145.68	0.39	138.38	125.00	0.37	115.30	105.72	0.3
21	154.93	138.74	0.37	131.79	119.05	0.35	109.81	100.69	0.3
22	147.89	132.44	0.36	125.80	113.64	0.33	104.82	96.11	0.3
23	141.46	126.68	0.34	120.33	108.70	0.32	100.26	91.93	0.3
24	135.56	121.40	0.33	115.31	104.17	0.31	96.08	88.10	0.2
25	130.14	116.55	0.31	110.70	100.00	0.29	92.24	84.58	0.2
26	125.14	112.06	0.30	106.44	96.16	0.28	88.69	81.32	0.2
27	120.50	107.91	0.29	102.50	92.60	0.27	85.41	78.31	0.2
28	116.20	104.06	0.28	98.84	89.29	0.26	82.36	75.52	0.2
29	112.19	100.47	0.27	95.43	86.21	0.25	79.52	72.91	0.2
30	108.45	97.12	0.26	92.25	83.34	0.24	76.87	70.48	0.2
31	104.95	93.99	0.25	89.27	80.65	0.24	74.39	68.21	0.2
32	101.67	91.05	0.25	86.48	78.13	0.23	72.06	66.08	0.2
33	98.59	88.29	0.24	83.86	75.76	0.22	69.88	64.07	0.2
34	95.69	85.70	0.23	81.40	73.53	0.22	67.82	62.19	0.
35	92.96	83.25	0.22	79.07	71.43	0.21	65.88	60.41	0.1
36	90.38	80.93	0.22	76.88	69.45	0.20	64.05	58.73	0.
37	87.93	78.75	0.21	74.80	67.57	0.20	62.32	57.15	0.
38	85.62	76.67	0.21	72.83	65.79	0.19	60.68	55.64	0.
39	83.42	74.71	0.20	70.96	64.10	0.19	59.13	54.22	0.
40	81.34	72.84	0.20	69.19	62.50	0.18	57.65	52.86	0.
41	79.35	71.06	0.19	67.50	60.98	0.18	56.24	51.57	0.
42	77.47	69.37	0.19	65.89	59.53	0.17	54.90	50.34	0.
43	75.66	67.76	0.18	64.36	58.14	0.17	53.63	49.17	0.
44	73.94	66.22	0.18	62.90	56.82	0.17	52.41	48.06	0.
45	72.30	64.75	0.17	61.50	55.56	0.16	51.24	46.99	0.
46	70.73	63.34	0.17	60.16	54.35	0.16	50.13	45.97	0.
47	69.22	61.99	0.17	58.88	53.19	0.16	49.06	44.99	0.
48	67.78	60.70	0.16	57.66	52.09	0.15	48.04	44.05	0.

Safe loads given include weight of beam
Maximum fiber-stress, 16 000 lb per sq in
The section-numbers are given for convenience in identification and order

Table XII (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Bethlehem Girder Beams

Beams secured against yielding sidewise

Span, ft	24-in G		Add for each lb increase in weight	20-in G		Add for each lb increase in weight	18-in G	Add for each lb increase in weight
	G24 a	G24		G20 a	G20		G18	
	140 lb	120 lb		140 lb	112 lb		92 lb	
12	155.61	133.60	0.52	130.43	104.09	0.44	78.59	0.39
13	143.64	123.33	0.48	120.40	96.09	0.40	72.54	0.36
14	133.38	114.52	0.45	111.80	89.23	0.37	67.36	0.34
15	124.48	106.88	0.42	104.34	83.28	0.35	62.87	0.31
16	116.71	100.20	0.39	97.82	78.07	0.33	58.94	0.29
17	109.84	94.31	0.37	92.07	73.48	0.31	55.47	0.28
18	103.74	89.07	0.35	86.95	69.40	0.29	52.39	0.26
19	98.28	84.38	0.33	82.38	65.74	0.28	49.63	0.25
20	93.37	80.16	0.31	78.26	62.46	0.26	47.15	0.24
21	88.92	76.35	0.30	74.53	59.48	0.25	44.91	0.22
22	84.88	72.88	0.29	71.14	56.78	0.24	42.87	0.21
23	81.19	69.71	0.27	68.05	54.31	0.23	41.00	0.20
24	77.80	66.80	0.26	65.22	52.05	0.22	39.29	0.20
25	74.69	64.15	0.25	62.61	49.97	0.21	37.72	0.19
26	71.82	61.66	0.24	60.20	48.04	0.20	36.27	0.18
27	69.16	59.38	0.23	57.97	46.26	0.19	34.93	0.17
28	66.69	57.26	0.22	55.90	44.61	0.19	33.68	0.17
29	64.39	55.29	0.22	53.97	43.07	0.18	32.52	0.16
30	62.24	53.44	0.21	52.17	41.64	0.17	31.43	0.16
31	60.24	51.72	0.20	50.49	40.30	0.17	30.42	0.15
32	58.35	50.10	0.20	48.91	39.04	0.16	29.47	0.15
33	56.58	48.58	0.19	47.43	37.85	0.16	28.58	0.14
34	54.92	47.15	0.18	46.04	36.74	0.15	27.74	0.14
35	53.35	45.81	0.18	44.72	35.69	0.15	26.94	0.13
36	51.87	44.54	0.17	43.48	34.70	0.15	26.20	0.13
37	50.47	43.33	0.17	42.30	33.76	0.14	25.49	0.13
38	49.14	42.19	0.17	41.19	32.87	0.14	24.82	0.12
39	47.88	41.11	0.16	40.13	32.03	0.13	24.18	0.12
40	46.68	40.08	0.16	39.13	31.23	0.13	23.58	0.12

Safe loads given include weight of beam. Maximum fiber-stress, 16 000 lb per sq in

Loads given above the heavy lines are greater than safe loads for web-crippling. See paragraphs and accompanying foot-note, page 567, relating to web-buckling of beams

Safe loads given below the lower, broken line cause deflections exceeding 1/360 of the span

The section-numbers are given for convenience in identification and ordering

Table XII (Continued). Safe Uniform Loads in Tons of 2 000 Pou[nds] for Bethlehem Girder Beams

Beams secured against yielding sidewise

Span, ft	15-in G			Add for each lb increase in weight	12-in G		A[dd for] e[ach lb] i[ncrease in] w[eight]
	G15 b	G15 a	G15		G12 a	G12	
	140 lb	104 lb	73 lb		70 lb	55 lb	
10	113.26	86.76	62.83	0.39	47.89	38.40	[illegible]
11	102.96	78.88	57.12	0.36	43.54	34.91	[illegible]
12	94.38	72.30	52.36	0.33	39.91	32.00	[illegible]
13	87.12	66.74	48.33	0.30	36.84	29.54	[illegible]
14	80.90	61.97	44.88	0.28	34.21	27.43	[illegible]
15	75.51	57.84	41.89	0.26	31.93	25.60	[illegible]
16	70.79	54.23	39.27	0.25	29.93	24.00	[illegible]
17	66.62	51.04	36.96	0.23	28.17	22.59	[illegible]
18	62.92	48.20	34.91	0.22	26.61	21.33	[illegible]
19	59.61	45.67	33.07	0.21	25.21	20.21	[illegible]
20	56.63	43.38	31.42	0.20	23.95	19.20	[illegible]
21	53.93	41.32	29.92	0.19	22.81	18.28	0.[illegible]
22	51.48	39.44	28.56	0.18	21.77	17.45	0.[illegible]
23	49.24	37.72	27.32	0.17	20.82	16.69	0.[illegible]
24	47.19	36.15	26.18	0.16	19.95	16.00	0.[illegible]
25	45.30	34.71	25.13	0.16	19.16	15.36	0.[illegible]
26	43.56	33.37	24.17	0.15	18.42	14.77	0.[illegible]
27	41.95	32.13	23.27	0.15	17.74	14.22	0.[illegible]
28	40.45	30.99	22.44	0.14	17.10	13.71	0.[illegible]
29	39.05	29.92	21.67	0.14	16.51	13.24	0.[illegible]
30	37.75	28.92	20.94	0.13	15.96	12.80	0.[illegible]
31	36.54	27.99	20.27	0.13	15.45	12.39	0[illegible]
32	35.39	27.11	19.63	0.12	14.97	12.00	0[illegible]
33	34.32	26.29	19.04	0.12	14.51	11.64	0[illegible]
34	33.31	25.52	18.48	0.12	14.09	11.29	0[illegible]
35	32.36	24.79	17.95	0.11	13.68	10.97	0[illegible]

Safe loads given include weight of beam. Maximum fiber-stress, 16 000 l[b per] sq in

Load given above the heavy line is greater than a safe load for web-crip[pling]. See paragraphs and accompanying foot-note, page 567, relating to web-buc[kling] of beams.

Safe loads given below the lower, broken lines cause deflections exceedin[g] of the span.

The section-numbers are given for convenience in identification and orde[ring]

Table XII (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Bethlehem Girder Beams

Beams secured against yielding sidewise

Span, ft	10-in G G10 44 lb	Add for each lb increase in weight	Span, ft	9-in G G9 38 lb	Add for each lb increase in weight	8-in G G8 32.5 lb	Add for each lb increase in weight
10	26.05	0.26	5	40.50	0.47	30.51	0.42
11	23.68	0.24	6	33.75	0.39	25.42	0.35
12	21.71	0.22	7	28.93	0.34	21.79	0.30
13	20.04	0.20	8	25.31	0.29	19.07	0.26
14	18.61	0.19	9	22.50	0.26	16.95	0.23
15	17.37	0.17	10	20.25	0.23	15.25	0.21
16	16.28	0.16	11	18.41	0.21	13.87	0.19
17	15.32	0.15	12	16.88	0.20	12.71	0.17
18	14.47	0.15	13	15.58	0.18	11.73	0.16
19	13.71	0.14	14	14.47	0.17	10.90	0.15
20	13.03	0.13	15	13.50	0.16	10.17	0.14
21	12.40	0.12	16	12.66	0.15	9.53	0.13
22	11.84	0.12	17	11.91	0.14	8.97	0.12
23	11.33	0.11	18	11.25	0.13	8.47	0.12
24	10.85	0.11	19	10.66	0.12	8.03	0.11
25	10.42	0.10	20	10.13	0.12	7.63	0.10
26	10.02	0.10	21	9.64	0.11	7.26	0.10
27	9.65	0.10	22	9.21	0.11	6.93	0.09
28	9.30	0.09	23	8.80	0.10	6.63	0.09
29	8.98	0.09	24	8.44	0.10	6.36	0.08
30	8.68	0.09	25	8.10	0.09	6.10	0.08
31	8.40	0.08	26	7.79	0.09	....	....
32	8.14	0.08	27	7.50	0.09	....	....
33	7.89	0.08	28	7.23	0.08	....	....
34	7.66	0.08	29	6.98	0.08	....	....
35	7.44	0.07	30	6.75	0.07	....	....

Safe loads given include weight of beam. Maximum fiber-stress, 16 000 lb per sq in

Loads given above the heavy lines are greater than safe loads for web-crippling. See paragraphs and accompanying foot-note, page 567, relating to web-buckling of beams

Safe loads given below the lower, broken lines cause deflections exceeding 1/360 of the span.

The section-numbers are given for convenience in identification and ordering

Table XIII. Safe Uniform Loads in Tons of 2 000 Pounds for Bethlehem I Beams

Beams secured against yielding sidewise

Span, ft	30-in I B30 120 lb	Add for each lb increase in weight	28-in I B28 105 lb	Add for each lb increase in weight	26-in I B26 90 lb	Add for each lb increase in weight
18	103.50	0.44	84.95	0.41	67.86	0.38
19	98.05	0.41	80.48	0.39	64.29	0.36
20	93.15	0.39	76.46	0.37	61.07	0.34
21	88.71	0.37	72.82	0.35	58.16	0.32
22	84.68	0.36	69.51	0.33	55.52	0.31
23	81.00	0.34	66.49	0.32	53.11	0.30
24	77.62	0.33	63.72	0.31	50.89	0.28
25	74.52	0.31	61.17	0.29	48.86	0.27
26	71.65	0.30	58.81	0.28	46.98	0.26
27	69.00	0.29	56.64	0.27	45.24	0.25
28	66.54	0.28	54.61	0.26	43.62	0.24
29	64.24	0.27	52.73	0.25	42.12	0.23
30	62.10	0.26	50.97	0.24	40.71	0.23
31	60.10	0.25	49.33	0.24	39.40	0.2
32	58.22	0.25	47.79	0.23	38.17	0.2
33	56.45	0.24	46.34	0.22	37.01	0.2
34	54.79	0.23	44.98	0.22	35.92	0.2
35	53.23	0.22	43.69	0.21	34.90	0.1
36	51.75	0.22	42.48	0.20	33.93	0.1
37	50.35	0.21	41.33	0.20	33.01	0.1
38	49.03	0.21	40.24	0.19	32.14	0.1
39	47.77	0.20	39.21	0.19	31.32	0.1
40	46.57	0.20	38.23	0.19	30.54	0.1
41	45.44	0.19	37.30	0.18	29.79	0.1
42	44.36	0.19	36.41	0.18	29.08	0.1
43	43.33	0.18	35.56	0.17	28.41	0.1
44	42.34	0.18	34.75	0.17	27.76	0.1
45	41.40	0.17	33.98	0.16	27.14	0.1
46	40.50	0.17	33.24	0.16	26.55	0.1
47	39.64	0.17	32.54	0.16	25.99	0.1
48	38.81	0.16	31.86	0.15	25.45	0.1

Safe loads given include weight of beam. Maximum fiber-stress, 16 000 lb sq in

The section-numbers are given for convenience in identification and orderi

Table XIII * (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Bethlehem I Beams

Beams secured against yielding sidewise

Span, ft	24-in I		Add for each lb increase in weight	20-in I					Add for each lb increase in weight
	B24 a	B24		B20 a		B20			
	84 lb	73 lb		82 lb	72 lb	69 lb	64 lb	59 lb	
12	88.22	77.45	0.52	69.33	65.18	56.40	54.32	52.10	0.44
13	81.43	71.49	0.48	63.99	60.17	52.06	50.14	48.09	0.40
14	75.62	66.38	0.45	59.42	55.87	48.34	46.56	44.65	0.37
15	70.58	61.96	0.42	55.46	52.14	45.12	43.45	41.68	0.35
16	66.16	58.08	0.39	51.99	48.88	42.30	40.74	39.07	0.33
17	62.27	54.67	0.37	48.94	46.01	39.81	38.34	36.77	0.31
18	58.81	51.63	0.35	46.22	43.45	37.60	36.21	34.73	0.29
19	55.72	48.91	0.33	43.78	41.17	35.62	34.31	32.90	0.28
20	52.93	46.47	0.31	41.60	39.11	33.84	32.59	31.26	0.26
21	50.41	44.26	0.30	39.61	37.25	32.23	31.04	29.77	0.25
22	48.12	42.24	0.29	37.81	35.55	30.76	29.63	28.42	0.24
23	46.03	40.41	0.27	36.17	34.01	29.42	28.34	27.18	0.23
24	44.11	38.72	0.26	34.66	32.59	28.20	27.16	26.05	0.22
25	42.35	37.17	0.25	33.28	31.29	27.07	26.07	25.01	0.21
26	40.72	35.74	0.24	32.00	30.08	26.03	25.07	24.04	0.20
27	39.21	34.42	0.23	30.81	28.97	25.07	24.14	23.15	0.19
28	37.81	33.19	0.22	29.71	27.93	24.17	23.28	22.33	0.19
29	36.50	32.05	0.22	28.69	26.97	23.34	22.48	21.56	0.18
30	35.29	30.98	0.21	27.73	26.07	22.56	21.73	20.84	0.17
31	34.15	29.98	0.20	26.84	25.23	21.83	21.03	20.17	0.17
32	33.08	29.04	0.20	26.00	24.44	21.15	20.37	19.54	0.16
33	32.08	28.16	0.19	25.21	23.70	20.51	19.75	18.94	0.16
34	31.14	27.33	0.19	24.47	23.00	19.90	19.17	18.39	0.15
35	30.25	26.55	0.18	23.77	22.35	19.34	18.62	17.86	0.15
36	29.41	25.82	0.17	23.11	21.73	18.80	18.11	17.37	0.15
37	28.61	25.12	0.17	22.48	21.14	18.29	17.62	16.90	0.14
38	27.86	24.46	0.17	21.89	20.58	17.81	17.15	16.45	0.14
39	27.14	23.83	0.16	21.33	20.06	17.35	16.71	16.03	0.13
40	26.47	23.23	0.16	20.80	19.55	16.92	16.30	15.63	0.13

Safe loads given include weight of beam. Maximum fiber-stress, 16 000 lb per sq in

Loads given above the heavy lines are greater than safe loads for web-crippling. See paragraphs and accompanying foot-note, page 567, relating to web-buckling of beams

The section-numbers are given for convenience in identification and ordering

* See data for Additional Sections, for 22-in and 24-in beams, in Pamphlet S-10, published March 1, 1921, by the Bethlehem Steel Company.

Table XIII * (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Bethlehem I Beams

Beams secured against yielding sidewise

Span, ft	18-in I B18 59 lb	18-in I B18 54 lb	18-in I B18 48.5 lb	Add for each lb increase in weight	15-in I B15 b 71 lb	15-in I B15 a 54 lb	15-in I B15 46 lb	15-in I B15 41 lb	15-in I B15 38 lb	Add each incre in weig
12	43.62	41.58	39.42	0.39	47.18	36.15	28.73	27.06	26.23	0.3
13	40.26	38.38	36.39	0.36	43.55	33.37	26.52	24.98	24.21	0.3
14	37.39	35.64	33.79	0.34	40.44	30.99	24.62	23.19	22.48	0.2
15	34.90	33.26	31.54	0.31	37.75	28.92	22.98	21.65	20.98	0.2
16	32.71	31.18	29.56	0.29	35.39	27.11	21.55	20.30	19.67	0.2
17	30.79	29.35	27.83	0.28	33.30	25.52	20.28	19.10	18.51	0.2
18	29.08	27.72	26.28	0.26	31.45	24.10	19.15	18.04	17.49	0.2
19	27.55	26.26	24.90	0.25	29.80	22.83	18.14	17.09	16.56	0.2
20	26.17	24.95	23.65	0.24	28.31	21.69	17.24	16.24	15.74	0.2
21	24.93	23.76	22.53	0.22	26.96	20.66	16.42	15.46	14.99	0.1
22	23.79	22.68	21.50	0.21	25.74	19.72	15.67	14.76	14.31	0.1
23	22.76	21.70	20.57	0.21	24.62	18.86	14.99	14.12	13.68	0.1
24	21.81	20.79	19.71	0.20	23.59	18.07	14.36	13.53	13.11	0.1
25	20.94	19.96	18.92	0.19	22.65	17.35	13.79	12.99	12.59	0.1
26	20.13	19.19	18.19	0.18	21.78	16.68	13.26	12.49	12.11	0.1
27	19.39	18.48	17.52	0.17	20.97	16.07	12.77	12.03	11.66	0.1
28	18.69	17.82	16.89	0.17	20.22	15.49	12.31	11.60	11.24	0.1
29	18.05	17.21	16.31	0.16	19.52	14.96	11.89	11.20	10.85	0.1
30	17.45	16.63	15.77	0.16	18.87	14.46	11.49	10.82	10.49	0.1
31	16.88	16.10	15.26	0.15	18.26	13.99	11.12	10.47	10.15	0.1
32	16.36	15.59	14.78	0.15	17.69	13.56	10.77	10.15	9.84	0.1
33	15.86	15.12	14.33	0.14	17.16	13.15	10.45	9.84	9.54	0.1
34	15.40	14.68	13.91	0.14	16.65	12.76	10.14	9.55	9.26	0.1
35	14.96	14.26	13.52	0.13	16.18	12.39	9.85	9.28	8.99	0.1
36	14.54	13.86	13.14	0.13	15.73	12.05	9.58	9.02	8.74	0.
37	14.15	13.49	12.78	0.13	15.30	11.72	9.32	8.78	8.51	0.
38	13.77	13.13	12.45	0.12	14.90	11.42	9.07	8.55	8.28	0.
39	13.42	12.79	12.13	0.12	14.52	11.12	8.84	8.33	8.07	0.
40	13.09	12.47	11.83	0.12	14.15	10.84	8.62	8.12	7.87	0.

Safe loads given include weight of beam. Maximum fiber-stress, 16 000 lb sq in

Load given above the heavy line is greater than safe load for web-crippl See paragraphs and accompanying foot-note, page 567, relating to web-buck of beams

Safe loads given below the broken lines cause deflections exceeding 1/360 of span

The section-numbers are given for convenience in identification and orderi

* See data for Additional Sections, for 18-in beams, in Pamphlet S-10, pub March 1, 1921, by the Bethlehem Steel Company.

Table XIII (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Bethlehem I Beams

Beams secured against yielding sidewise

Span, ft	12-in I			Add for each lb increase in weight	10-in I		Add for each lb increase in weight
	B12 *a*	B12			B10		
	36 lb	32 lb	28.5 lb		28.5 lb	23.5 lb	
9	26.59	22.57	21.36	0.35	15.95	14.57	0.29
10	23.93	20.31	19.22	0.31	14.35	13.11	0.26
11	21.76	18.46	17.47	0.29	13.05	11.92	0.24
12	19.94	16.92	16.02	0.26	11.96	10.92	0.22
13	18.41	15.62	14.79	0.24	11.04	10.08	0.20
14	17.09	14.51	13.73	0.22	10.25	9.36	0.19
15	15.95	13.54	12.81	0.21	9.57	8.74	0.17
16	14.96	12.69	12.01	0.20	8.97	8.19	0.16
17	14.08	11.95	11.31	0.19	8.44	7.71	0.15
18	13.30	11.28	10.68	0.17	7.97	7.28	0.15
19	12.60	10.69	10.12	0.17	7.55	6.90	0.14
20	11.97	10.15	9.61	0.16	7.18	6.55	0.13
21	11.40	9.67	9.15	0.15	6.84	6.24	0.12
22	10.88	9.23	8.74	0.14	6.52	5.96	0.12
23	10.41	8.83	8.36	0.14	6.24	5.70	0.11
24	9.97	8.46	8.01	0.13	5.98	5.46	0.11
25	9.57	8.12	7.69	0.13	5.74	5.24	0.10
26	9.20	7.81	7.39	0.12	5.52	5.04	0.10
27	8.86	7.52	7.12	0.12	5.32	4.86	0.10
28	8.55	7.25	6.86	0.11	5.13	4.68	0.09
29	8.25	7.00	6.63	0.11	4.95	4.52	0.09
30	9.98	6.77	6.41	0.11	4.78	4.37	0.09
31	7.72	6.55	6.20	0.10	....	....	....
32	7.48	6.35	6.01	0.10	....	....	....
33	7.25	6.15	5.82	0.10	....	....	....
34	7.04	5.97	5.65	0.09	....	....	....
35	6.84	5.80	5.49	0.09	....	....	....

Safe loads given include weight of beam. Maximum fiber-stress, 16 000 lb per sq in

Safe loads given below the broken lines cause deflections exceeding 1/360 of the span

The section-numbers are given for convenience in identification and ordering

Table XIII (Continued). Safe Uniform Loads in Tons of 2 000 Pound[s] for Bethlehem I Beams

Beams secured against yielding sidewise

Span, ft	9-in I		Add for each lb increase in weight	8-in I		Add f[or] each l[b] increa[se] in weig[ht]
	B9			B8		
	24 lb	20 lb		19.5 lb	17.5 lb	
5	21.83	20.18	0.47	16.16	15.30	0.42
6	18.19	16.81	0.39	13.46	12.75	0.35
7	15.60	14.41	0.34	11.54	10.93	0.30
8	13.65	12.61	0.29	10.10	9.57	0.26
9	12.13	11.21	0.26	8.98	8.50	0.23
10	10.92	10.09	0.24	8.08	7.65	0.21
11	9.92	9.17	0.21	7.34	6.96	0.19
12	9.10	8.41	0.20	6.73	6.38	0.17
13	8.40	7.76	0.18	6.21	5.89	0.16
14	7.80	7.21	0.17	5.77	5.47	0.15
15	7.28	6.73	0.16	5.39	5.10	0.14
16	6.82	6.31	0.15	5.05	4.78	0.13
17	6.42	5.93	0.14	4.75	4.50	0.12
18	6.07	5.61	0.13	4.49	4.25	0.12
19	5.75	5.31	0.13	4.25	4.03	0.11
20	5.46	5.04	0.12	4.04	3.83	0.11
21	5.20	4.80	0.11	3.85	3.64	0.10
22	4.96	4.59	0.11	3.67	3.48	0.10
23	4.75	4.39	0.10	3.51	3.33	0.09
24	4.55	4.20	0.10	3.37	3.19	0.09
25	4.37	4.04	0.10	3.23	3.06	0.08
26	4.20	3.88	0.09	....	....	....
27	4.04	3.74	0.09	....	....	....
28	3.90	3.60	0.09	....	....	....
29	3.76	3.48	0.08	....	....	....
30	3.64	3.36	0.08	....	....	....

Safe loads given include weight of beam. Maximum fiber-stress, 16 000 lb [per] sq in

Safe loads given below the broken lines cause deflections exceeding 1/360 of span

The section-numbers are given for convenience in identification and orderi[ng]

...ed Single-Beam and Double-Beam Girders.* Where a SINGLE ...LE-BEAM is insufficient to carry a load, the required capacity may be secured ...brication in various ways. TWO BEAMS can be used, connected by bolts ...separators. The total strength of these is twice that of the SINGLE BEAM ...he same depth and weight. Care should be taken, however, to see that ...loads are apportioned to them equally, and where it is necessary for the ...s to act as a unit, the separators should consist of plates and angles and not ...ade of cast iron. If the loading is not uniformly distributed over the two ...s, the strength of each must be computed separately. The use of a SINGLE-...M GIRDER with plates at top and bottom to sustain a given load is often more ...omical in material than the use of TWO BEAMS connected by bolts and ...rators. The beam girders in Table XIV, pages 605–6, have about twice ...carrying capacity of the single beams of which they are built.

...ables XIV and XV give the SAFE LOADS for the SINGLE AND DOUBLE-BEAM ...ERS commonly used. The values given in the tables are founded upon ...moments of inertia of the various sections, deductions being made for the ...t-holes in both flanges. In Table XIV, taken by permission from Carnegie's ...ket Companion, the safe loads are based upon a fiber-stress for flange-bend... of 16 000 lb per sq in, and in Table XV, retained from the former edition of ...der's Pocket-Book, upon a fiber-stress of 13 000 lb per sq in. For other fiber-...ses, as 14 000 or 15 000 lb per sq in, the safe loads in Tables XIV or XV ...y be decreased or increased by PROPORTION as the LOADS vary as the FIBER-...ESSES.†

* For tables of riveted plate girders, see Chapter XX.

† The editors decided to retain Table XV for the safe uniformly distributed loads for ...ed steel-beam box girders, based upon a bending fiber-stress of 13 000 lb per sq in. ...se this table for fiber-stresses of 14 000, 15 000 or 16 000 lb per sq in, divide the ...ar loads by 13 and multiply the quotients by 14, 15 or 16, respectively, for the safe ...s at the required fiber-stress. In regard to Table XV, Mr. Kidder said, in the ...ding editions of this pocket-book, "in order to amply compensate for the deteriora-... of the metal around the rivet-holes from punching, and also because these girders ...re often used to support permanent loads, such as brick or stone walls, the maximum ...-stress [for riveted double-beam girders] was limited to 13 000 lb per sq in, although ...but right to state that most of the latest handbooks of the steel-manufacturers give ...s of safe loads for such girders based upon a fiber-stress of 15 000 lb per sq in. ...author advises that for loads of masonry, which usually come very close to the ...ated loads, and which are constantly applied, the girders be not loaded beyond the ...s given in the following tables (that is, based upon 13 000 lb per sq in), while for ...ary floor-loads, which seldom reach the estimated loads, an addition of ⅙th may be ...d to the values given in the tables."

...ders fabricated of single steel I beams and plates riveted to the upper and lower ..., as shown in Table XIV, are not often used to support masonry walls, because of ...relatively narrow flange-width and lack of lateral stiffness. In case they are used ...port masonry walls and are not thoroughly braced laterally, it is recommended that ...afe loads be reduced as explained, from those given in Table XIV, to agree with ...r-stress of 13 000 or 14 000 lb per sq in, according to the span, bracing, character ...ding, etc. It is recommended, also, that for girders fabricated of two steel I beams ...plates riveted to the flanges, as shown in Table XV, and carrying masonry walls, the ...loads, given in this table and computed for a fiber-stress of 13 000 lb per sq in, be used, ...increased, that the fiber-stress be taken not greater than 14 000 lb per sq in.

...ent handbooks have contained tables of safe uniformly distributed loads for fabri-...d steel girders computed from safe unit fiber-stresses, in pounds per square inch, ...flange-bending as follows. For RIVETED SINGLE-BEAM GIRDERS: Carnegie Steel ...any, 1903 Edition, no tables; Carnegie, 1915 Edition, 16 000, based upon the ...n-modulus of the gross area of the cross-section; Cambria Steel Company, 1912 ...ion, no tables; (former) Passaic Steel Company, 1903 Edition, no tables; Kidder's ...ket-Book, previous editions, no tables. For RIVETED DOUBLE-BEAM GIRDERS: Car-

Example 21. A 13-in brick wall, 15 ft high, is to be built over an openin
24 ft. What is the size of the double-beam girder required?

Solution. Assuming 25 ft as the distance, center to center of bearings a d
lb per cu ft as the weight of brickwork, the weight of the wall is 25 × 15 ×
45 375 lb, or about 22.68 tons. From Table XV, page 610, a girder compos
two 12-in steel beams, each weighing 31.5 lb per ft, and two 14 by ½-in fl
plates will carry safely, for a span of 25 ft, a uniformly distributed load of
tons, including its own weight. Deducting the latter, 1.42 tons, given i
next column, the result is 21.81 tons for the safe net load, which is 0.87 to
than required. From the following column of the table it is seen that b
creasing the thickness of the flange-plates 1/16 in it is safe to add 1.52 tons t
allowable load. This will more than make up the difference. Hence th
quired DOUBLE-BEAM GIRDER will be composed of two 12-in 31.5-lb b
and two 14 by 9/16-in steel flange-plates.

A SINGLE-BEAM GIRDER (according to Table XIV, page 606), compos
one 15-in 42-lb I beam and two 8 by ½-in flange-plates will carry, at 1
lb per sq ft, 49 000 lb over a span of 25 ft, and as it is lighter, weighin
69.2 lb per ft to the others 113.6 lb, it would be more economical.
DOUBLE-BEAM GIRDER is, however, more suitable in this particular case,
13-in wall should have a wider bearing than 8 in, and, also, the safe load s
be decreased from the tabular load to correspond to a fiber-stress of 13 c
14 000 lb per sq in because of the nature of the loading, the long span,
or, what amounts to the same thing, the strength of the girder shou
increased to correspond to the decreased fiber-stress. (See foot-note, page
A 49 000-lb load at 16 000 lb per sq in fiber-stress corresponds to a 49
16/13 = 60 307-lb load at 13 000 fiber-stress, as far as selecting a correspo
girder from table is concerned. A SINGLE-BEAM GIRDER (Table XIV)
posed of one 15-in 60-lb I beam and two 9 by ⅝-in flange-plates will
68 000 lb and weighs only 98.3 lb per ft. Therefore, as far as strength i
cerned, to suit the conditions of loading, this would be the proper SINGLE-
GIRDER to use, and it would also be cheaper than the DOUBLE-BEAM G
determined by Table XV; but the width of bearing for the 13-in wall i
only 9 in compared to 14 in with the DOUBLE-BEAM GIRDER.

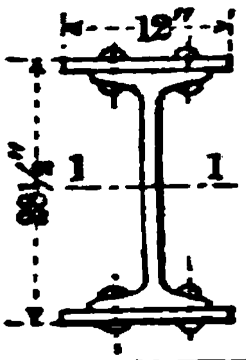
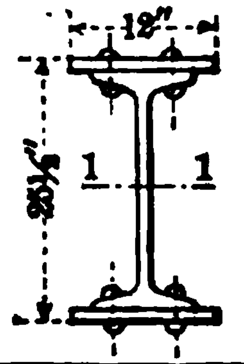
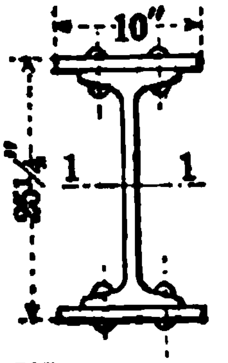
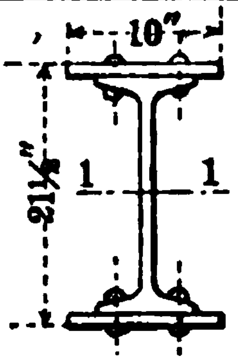
negie, 1903, 15 000, 13/16-in rivet-holes deducted; Carnegie, 1915, no tables; Ca
15 000, 13/16-in rivet-holes deducted; Passaic, 15 000, 13/16 or 15/16-in rivet-holes ded
Kidder, previous and new editions, 13 000, 15/16-in rivet-holes deducted. For RI
SINGLE-WEB, PLATE-AND-ANGLE GIRDERS (see Chapter XX): Carnegie, 1903, 1
13/16-in rivet-holes deducted; Carnegie, 1915, 16 000, based upon section-modulus
gross area of cross-section; Cambria, 15 000, 13/16 or 15/16-in rivet-holes ded
Passaic, 15 000, 13/16 or 15/16-in rivet-holes deducted; Kidder, previous editions,
and 13 000 for flanges, 13/16 or 15/16-in rivet-holes deducted (also contained the F
tables). For RIVETED MULTIPLE-WEB, PLATE-AND-ANGLE GIRDERS (see Chapter
Carnegie, 1903, 15 000, 13/16-in rivet-holes deducted; Carnegie, 1915, 16 000, based
section-modulus of the gross area of cross-section (the elements, only, of these g
and not the loads, are given); Cambria, no tables; Passaic, 15 000, 13/16 or 1
rivet-holes, deducted; Kidder, previous editions, same as for single-web plat
girders.

The revised edition of Kidder's Handbook uses, by permission, the Carnegie
for all but the riveted double-beam girders, for which the old Kidder tables are re

The limiting conditions of use are fully explained in the text and foot-notes. E
in-chief.

Table XIV.* Safe Uniform Loads in Units of 1 000 Pounds for Riveted Steel-Beam Girders

Maximum bending stress, 16 000† lb per sq in

Span, ft	 27-in 90-lb beam 12 by ¾-in plates		 24-in 79.9-lb beam, 12 by ¾-in plates		 24-in 79.9-lb beam, 10 by ⅝-in plates		 20-in 79.9-lb beam, 10 by ¾-in plates		Coefficients of deflection
	Safe loads	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Safe loads	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Safe loads	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Safe loads	Increase in safe loads for 1/16-in increase in thickness of flange-plates	
13	370	15.9	312	14.2	259	11.7	235	9.7	2.80
14	343	14.8	289	13.2	240	10.9	218	9.0	3.24
15	321	13.8	270	12.3	224	10.1	204	8.4	3.72
16	301	13.0	253	11.5	210	9.5	191	7.9	4.24
17	283	12.2	238	10.9	198	9.0	180	7.4	4.78
18	267	11.5	225	10.3	187	8.4	170	7.0	5.36
19	253	10.9	213	9.7	177	8.0	161	6.6	5.98
20	240	10.4	203	9.2	168	7.6	153	6.3	6.62
21	229	9.9	193	8.8	160	7.2	146	6.0	7.30
22	219	9.4	184	8.4	153	6.9	139	5.7	8.01
23	209	9.0	176	8.0	146	6.6	133	5.5	8.76
24	200	8.6	169	7.7	140	6.3	127	5.3	9.53
25	192	8.3	162	7.4	135	6.1	122	5.0	10.35
26	185	8.0	156	7.1	129	5.9	118	4.8	11.19
27	178	7.7	150	6.8	125	5.6	113	4.7	12.07
28	172	7.4	145	6.6	120	5.4	109	4.5	12.98
29	166	7.1	140	6.4	116	5.2	105	4.3	13.92
30	160	6.9	135	6.2	112	5.1	102	4.2	14.90
31	155	6.7	131	6.0	109	4.9	99	4.1	15.91
32	150	6.5	127	5.8	105	4.8	96	3.9	16.95
33	146	6.3	123	5.6	102	4.6	93	3.8	18.03
34	141	6.1	119	5.4	99	4.5	90	3.7	19.13
35	137	5.9	116	5.3	96	4.3	87	3.6	20.28
Area	44.33 in²		41.33 in²		35.83 in²		38.73 in²		....
S_{1-1}	450.8 in³		380.0 in³		315.5 in³		286.7 in³		....
Weight	152.2 lb per ft		141.1 lb per ft		122.4 lb per ft		132.4 lb per ft		....

Safe loads above the heavy, horizontal lines exceed the resistance of the web and girders should be provided with stiffeners; for limiting conditions, see explanatory notes on page 567. See Pocket Companion for 13 and 14-ft spans.
Weights given for girders do not include stiffeners, rivet-heads or other details

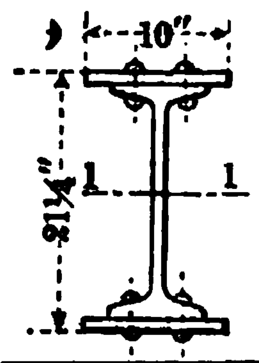
* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

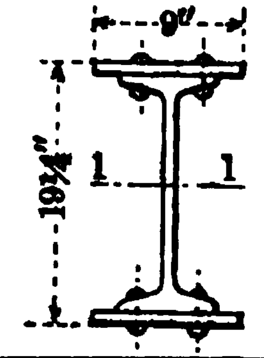
† See paragraph on Riveted Single-Beam and Double-Beam Girders, page 603, and the note for same regarding fiber-stresses.

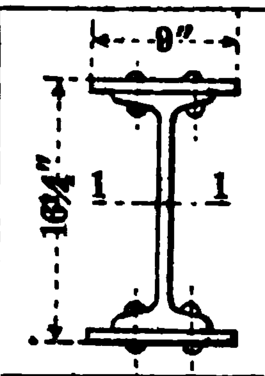
S is the section-modulus or section-factor of the cross-section with reference to the axis 1-1.

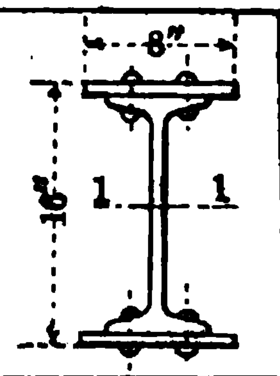
Table XIV* (Continued). Safe Uniform Loads in Units of 1 000 Pounds for Riveted Steel-Beam Girders

Maximum bending stress, 16 000† lb per sq in









Span, ft	20-in 65.4-lb beam, 10 by ⅝-in plates		18-in 54.7-lb beam, 9 by ⅝-in plates		15-in 60.8-lb beam, 9 by ⅝-in plates		15-in 42.9-lb beam, 8 by ½-in plates	
	Safe loads	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Safe loads	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Safe loads	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Safe loads	Increase in safe loads for 1/16-in increase in thickness of flange-plates
9	279	14.2	218	11.5	189	9.4	137	8.5
10	251	12.7	196	10.3	170	8.5	123	7.6
11	228	11.6	178	9.4	155	7.7	112	6.9
12	209	10.6	164	8.6	142	7.1	102	6.4
13	193	9.8	151	7.9	131	6.5	95	5.9
14	179	9.1	140	7.4	122	6.1	88	5.5
15	167	8.5	131	6.9	113	5.7	82	5.1
16	157	8.0	123	6.5	106	5.3	77	4.8
17	148	7.5	115	6.1	100	5.0	72	4.5
18	139	7.1	109	5.7	95	4.7	68	4.2
19	132	6.7	103	5.4	90	4.5	65	4.0
20	125	6.4	98	5.2	85	4.3	61	3.8
21	119	6.1	93	4.9	81	4.0	59	3.6
22	114	5.8	89	4.7	77	3.9	56	3.5
23	109	5.5	85	4.5	74	3.7	53	3.3
24	105	5.3	82	4.3	71	3.5	51	3.2
25	100	5.1	79	4.1	68	3.4	49	3.1
26	97	4.9	76	4.0	65	3.3	47	2.9
27	93	4.7	73	3.8	63	3.1	46	2.8
28	90	4.6	70	3.7	61	3.0	44	2.7
29	87	4.4	68	3.6	59	2.9	42	2.6
30	84	4.2	65	3.4	57	2.8	41	2.5
Area	31.58 in²		27.19 in²		28.93 in²		20.49 in²	
I/c_{1-1}‡	235.2 in³		184.1 in³		159.5 in³		115.3 in³	
Wgt	107.9 lb per ft		93.0 lb per ft		99.1 lb per ft		70.1 lb per ft	

Safe loads above the heavy, horizontal lines exceed the resistance of the web; girders should be provided with stiffeners; for limiting conditions, see explan notes on page 567. See Pocket Companion, 1915 for 9-ft. span

Weights given for girders do not include stiffeners, rivet-heads or other d

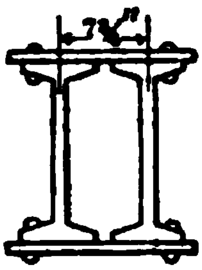
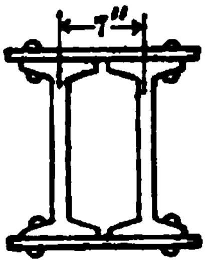
* From Pocket Companion, Carnegie Steel Company, Pitsburgh, Pa.

† See paragraph on Riveted Single-Beam and Double-Beam Girders, page 603, foot-note for same regarding fiber-stresses.

‡ I/c is the section-modulus or section-factor of the cross-section with reference axis 1–1.

Table XV. Safe Uniform Loads in Tons of 2 000 Pounds for Riveted Steel-Beam Box Girders

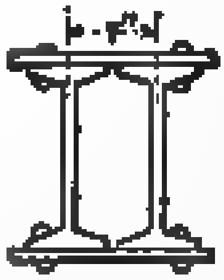
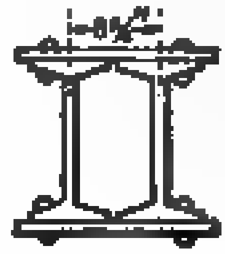
Two 20-in steel I beams and two 16 by 3/4-in steel plates

Dis-tance, center to center of bearings, ft	Two steel plates, 16 by 3/4 in  Two 20-in beams, 80.4 lb per ft			Two steel plates, 16 by 3/4 in  Two 20-in beams, 65.04 lb per ft			Increase in weight of girder for 1/16-in increase in thickness of flange-plates
	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Increase in safe loads for 1/16-in increase in thickness of flange-plates	
10	199.67	1.22	7.22	176.72	1.06	7.34	0.03
11	181.51	1.34	6.56	160.66	1.16	6.68	0.04
12	166.39	1.46	6.02	147.26	1.27	6.12	0.04
13	153.60	1.58	5.56	135.95	1.37	5.65	0.04
14	142.64	1.70	5.16	126.24	1.48	5.25	0.05
15	133.12	1.83	4.81	117.82	1.58	4.90	0.05
16	124.80	1.95	4.51	110.45	1.69	4.59	0.05
17	117.47	2.07	4.25	103.96	1.79	4.32	0.06
18	110.94	2.19	4.01	98.18	1.90	4.08	0.06
19	105.10	2.31	3.80	93.01	2.01	3.86	0.06
20	99.83	2.43	3.61	88.36	2.11	3.67	0.07
21	95.08	2.56	3.44	84.15	2.22	3.50	0.07
22	90.77	2.68	3.28	80.33	2.32	3.34	0.07
23	86.82	2.80	3.14	76.84	2.43	3.19	0.08
24	83.20	2.92	3.01	73.64	2.53	3.06	0.08
25	79.87	3.04	2.89	70.69	2.64	2.94	0.08
26	76.80	3.16	2.78	67.97	2.75	2.82	0.09
27	73.96	3.29	2.68	65.46	2.85	2.72	0.09
28	71.32	3.41	2.58	63.12	2.96	2.62	0.09
29	68.86	3.53	2.49	60.94	3.06	2.53	0.10
[illegible]	66.56	3.65	2.41	58.91	3.17	2.45	0.10
[illegible]	64.41	3.77	2.33	57.01	3.27	2.37	0.10
[illegible]	62.41	3.89	2.26	55.22	3.38	2.29	0.11
[illegible]	60.51	4.02	2.19	53.56	3.48	2.22	0.11
[illegible]	58.73	4.14	2.12	51.98	3.59	2.16	0.11
[illegible]	57.05	4.26	2.06	50.50	3.70	2.10	0.12
[illegible]	55.46	4.38	2.01	49.09	3.80	2.04	0.12
[illegible]	53.96	4.50	1.95	47.77	3.91	1.98	0.12
[illegible]	52.54	4.62	1.90	46.51	4.01	1.93	0.13
[illegible]	51.20	4.75	1.85	45.32	4.12	1.88	0.13

...ve values are based on a maximum fiber-stress of 13 000 lb per sq in, rivet-holes ...flanges deducted. See paragraph on Riveted Single-Beam and Double-Beam ...page 603, and the foot-note for same regarding fiber-stresses. Weights of girders ...d to lengths, center to center of bearings.

Table XV (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Riveted Steel-Beam Box Girders

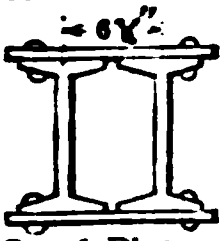
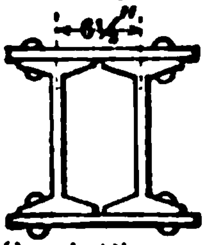
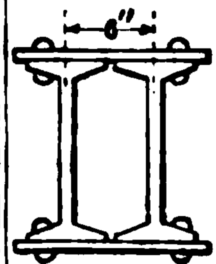
Two 18-in steel I beams and two 16 by ¾-in steel plates

Distance, center to center of bearings, ft	 Two 18-in beams, 70 lb per ft; Two 16 by ¾-in steel plates			Two 18-in beams, 54.7 lb per ft  Two 16 by ¾-in steel plates				Ad... we... of... cr... ¼... inc... th... ne... pl...
	Safe loads in tons, including weight of girder	Weight of girder, lb	Add to safe loads for 1/16-in increase in thickness of plates	Safe loads in tons, including weight of girder	Weight of girder, lb	Add to safe loads for 5 pounds increase in weight of beam	Add to safe loads for 1/16-in increase in thickness of plates	
12	132.2	2 712	5.43	123.0	2 352	2.81		
13	122.0	2 933	5.01	113.5	2 548	2.61		
14	113.3	3 164	4.66	105.3	2 744	2.43		
15	105.7	3 390	4.35	98.3	2 940	2.27		
16	99.1	3 616	4.07	92.2	3 136	2.12		
17	93.3	3 842	3.83	86.8	3 332	2.00		
18	88.1	4 068	3.62	82.0	3 528	1.90		
19	83.5	4 294	3.43	77.6	3 724	1.80		
20	79.3	4 520	3.26	73.8	3 920	1.70		
21	75.5	4 746	3.10	70.2	4 116	1.62		
22	72.1	4 972	2.96	67.0	4 312	1.54		
23	69.0	5 198	2.83	64.1	4 508	1.47		
24	66.1	5 424	2.72	61.5	4 704	1.41		
25	63.5	5 650	2.61	59.0	4 900	1.36		
26	61.0	5 876	2.51	56.7	5 096	1.30		
27	58.8	6 102	2.41	54.6	5 292	1.26		
28	56.6	6 328	2.33	52.7	5 488	1.21		
29	54.7	6 554	2.25	50.9	5 684	1.17		
30	52.9	6 780	2.17	49.2	5 880	1.13		
31	51.8	7 006	2.10	47.6	6 076	1.10		
32	49.6	7 232	2.04	46.1	6 272	1.06		
33	48.1	7 458	1.98	44.7	6 468	1.03		
34	46.7	7 684	1.92	43.4	6 664	1.00		
35	45.3	7 910	1.86	42.1	6 860	0.97		
36	44.1	8 136	1.81	41.0	7 056	0.94		
37	42.9	8 362	1.76	39.9	7 252	0.92		
38	41.2	8 588	1.72	38.8	7 448	0.90		

The above values are based on a maximum fiber-stress of 13 000 lb per sq in, ri... in both flanges deducted. See paragraph on Riveted Single-Beam and Doubl... Girders, page 603, and the foot-note for same regarding fiber-stresses. Weights ... correspond to lengths, center to center of bearings.

Table XV (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Riveted Steel-Beam Box Girders

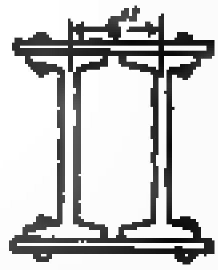
Two 15-in steel I beams and two 14 by ⅝-in steel plates

Dis- ance, enter to enter f bear- ings, ft	Two 15-in beams, 75.0 lb per ft  Steel Plates, 14 by ⅝ in		Two 15-in beams, 60.8 lb per ft  Steel Plates, 14 by ⅝ in		 Two 15-in beams, 42.9 lb per ft Steel Plates, 14 by ⅝ in		Increase in safe load for 1/16-in increase in thickness of flange-plates	Incre in weig of gir er fo 1/16-i increa in thic ness o flange plate
	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb		
10	122.33	1.06	111.01	0.91	90.29	0.72	4.63	0.03
11	111.21	1.17	100.92	1.00	82.08	0.79	4.21	0.03
12	101.95	1.27	92.51	1.09	75.24	0.86	3.86	0.03
13	94.10	1.38	85.40	1.18	69.45	0.93	3.57	0.04
14	87.38	1.48	79.30	1.27	64.50	1.00	3.31	0.04
15	81.56	1.59	74.01	1.36	60.19	1.08	3.09	0.04
16	76.46	1.70	69.38	1.45	56.43	1.15	2.90	0.05
17	71.96	1.80	65.30	1.54	53.11	1.22	2.72	0.05
18	67.96	1.91	61.67	1.63	50.16	1.29	2.57	0.05
19	64.39	2.01	58.43	1.72	47.52	1.36	2.43	0.05
20	61.17	2.12	55.50	1.81	45.14	1.44	2.32	0.06
21	58.25	2.22	52.86	1.90	42.99	1.51	2.21	0.06
22	55.60	2.33	50.46	2.00	41.04	1.58	2.11	0.06
23	53.19	2.43	48.27	2.09	39.25	1.65	2.02	0.07
24	50.97	2.54	46.25	2.18	37.62	1.72	1.93	0.07
[illegible]	48.93	2.65	44.40	2.27	36.12	1.79	1.85	0.07
[illegible]	47.05	2.76	42.70	2.36	34.72	1.87	1.78	0.08
[illegible]	45.31	2.86	41.12	2.45	33.44	1.94	1.71	0.08
[illegible]	43.69	2.96	39.65	2.54	32.25	2.01	1.66	0.08
[illegible]	42.18	3.07	38.28	2.63	31.13	2.08	1.60	0.08
[illegible]	40.78	3.17	37.00	2.72	30.09	2.15	1.54	0.09
[illegible]	39.46	3.28	35.81	2.81	29.12	2.23	1.49	0.09
[illegible]	38.23	3.38	34.69	2.80	28.21	2.30	1.45	0.09
[illegible]	37.07	3.46	33.64	2.99	27.36	2.37	1.41	0.10
[illegible]	35.98	3.60	32.65	3.08	26.55	2.44	1.37	0.10
[illegible]	34.95	3.70	31.72	3.17	25.80	2.51	1.33	0.10
[illegible]	33.98	3.81	30.84	3.27	25.08	2.58	1.29	0.10
[illegible]	33.06	3.91	30.00	3.36	24.40	2.66	1.25	0.11
[illegible]	32.20	4.02	29.21	3.45	23.76	2.73	1.22	0.11
[illegible]	31.37	4.13	28.47	3.54	23.15	2.80	1.19	0.11

above values are based on a maximum fiber-stress of 13 000 lb per sq in, rivet-ho… flanges deducted. See paragraph on Riveted Single-Beam and Double-Bea… page 603, and the foot-note for same regarding fiber-stresses. Weights of gird… …d to lengths, center to center of bearings.

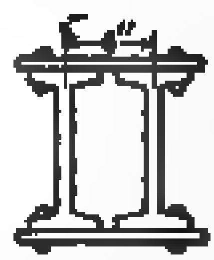
Table XV (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Riveted Steel-Beam Box Girders

Two 12-in steel I beams and two 14 by ½-in steel plates



Two 12-in beams, 40.8 lb per ft

Two steel plates, 14 by ½ in



Two 12-in beams, 31.8 lb per ft

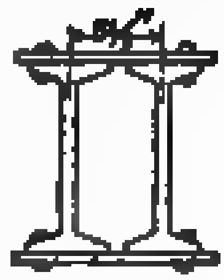
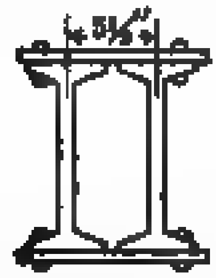
Two steel plates, 14 by ½ in

Distance, center to center of bearings, ft	Two 12-in beams, 40.8 lb per ft; Two steel plates, 14 by ½ in: Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Two 12-in beams, 31.8 lb per ft; Two steel plates, 14 by ½ in: Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Increa[se] in weig[ht] of gird[er] for 1/16[-in] increa[se] in thi[ck]ness [of] flang[e-] plat[es]
10	64 94	0 65	3 75	58 08	0.57	3 81	0.
11	59 02	0 71	3 40	52 80	0.63	3 45	0.
12	54.12	0.78	3 12	48 40	0.68	3.17	0.
13	49 95	0 84	2.88	44 68	0.74	2.93	0.
14	46 39	0 91	2 68	41 48	0.80	2.72	0
15	43 29	0.97	2 50	38 72	0.85	2 53	0
16	40.59	1 04	2 34	36 30	0.91	2.38	0
17	38 20	1.10	2 21	34 16	0.97	2 24	0
18	36.08	1.17	2.08	32 27	1 03	2.11	0
19	34.18	1.23	1 97	30 57	1.08	2 00	0
20	32.47	1.30	1 87	29 04	1 14	1.90	[illegible]
21	30.93	1.36	1 78	27.66	1 20	1 81	[illegible]
22	29.52	1 43	1.70	26 40	1.25	1.73	[illegible]
23	28.23	1.49	1 63	25 25	1.31	1 65	[illegible]
24	27.06	1.56	1.56	24.20	1 37	1.58	[illegible]
25	25 98	1 62	1 50	23 23	1 42	1.52	[illegible]
26	24.98	1.69	1 44	22 34	1 48	1.46	[illegible]
27	24 05	1.75	1.38	21.51	1.54	1.41	
28	23.19	1 82	1.34	20 74	1 60	1.36	
29	22.39	1 88	1.29	20.03	1.65	1.31	
30	21.65	1.95	1.25	19.36	1.71	1.27	
31	20.95	2.01	1 21	18.73	1.77	1.23	
[redacted]	20.29	2.08	1.17	18.15	1.82	1.19	
33	19.68	2.14	1.14	17.60	1.88	1.15	
34	19.10	2.21	1 10	17.08	1 94	1.12	
35	18.55	2.27	1.07	16 59	1 99	1.09	
36	18.04	2.34	1 04	16.13	2.05	1.06	
37	17.55	2.40	1.01	15 70	2.11	1.03	
38	17.09	2.47	0 99	15.28	2.17	1.00	
39	16.65	2.53	0 96	14.89	2.22	0.98	

The above values are based on a maximum fiber-stress of 13 000 lb per sq in., th[e] in both flanges deducted. See paragraph on Riveted Single-Beam and Doub[le] Girders, page 603, and the foot-note for same regarding fiber-stresses. Weights c[orrespond]... correspond to lengths, center to center of bearings.

Table XV (Continued). Safe Uniform Loads in Tons of 2 000 Pounds for Riveted Steel-Beam Box Girders

Two 10-in steel I Beams and two 12 by ½-in steel plates

Distance, center to center of bearings, ft	 Two 10-in beams, 35.0 lb per ft. Two steel plates, 12 by ½ in			 Two 10-in beams, 25.4 lb per ft. Two steel plates, 12 by ½ in			Increase in weight of girder for 1/16-in increase in thickness of flange-plates
	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Increase in safe loads for 1/16-in increase in thickness of flange-plates	Safe loads, uniformly distributed (including weight of girder), in tons of 2 000 lb	Weight of girder (including rivet-heads), in tons of 2 000 lb	Increase in safe loads for 1/16-in increase in thickness of flange-plates	
10	44.35	0.55	2.59	39.23	0.47	2.64	0.02
11	40.32	0.60	2.36	35.66	0.52	2.40	0.03
12	36.96	0.65	2.16	32.69	0.56	2.20	0.03
13	34.12	0.71	1.99	30.18	0.61	2.03	0.03
14	31.68	0.76	1.85	28.02	0.66	1.89	0.03
15	29.57	0.82	1.73	26.15	0.71	1.76	0.04
16	27.72	0.87	1.62	24.52	0.75	1.65	0.04
17	26.09	0.93	1.52	23.08	0.80	1.55	0.04
18	24.64	0.98	1.44	21.79	0.85	1.47	0.04
19	23.34	1.04	1.36	20.65	0.89	1.39	0.05
20	22.18	1.09	1.30	19.62	0.94	1.32	0.05
21	21.12	1.15	1.23	18.68	0.99	1.26	0.05
22	20.16	1.20	1.18	17.83	1.04	1.20	0.05
23	19.28	1.26	1.13	17.06	1.08	1.15	0.06
24	18.48	1.31	1.08	16.35	1.13	1.10	0.06
25	17.74	1.36	1.04	15.69	1.18	1.06	0.06
26	17.06	1.42	1.00	15.09	1.22	1.02	0.06
27	16.43	1.47	0.96	14.53	1.27	0.98	0.07
28	15.84	1.53	0.93	14.01	1.32	0.94	0.07
29	15.29	1.58	0.89	13.53	1.37	0.91	0.07
30	14.78	1.64	0.86	13.08	1.41	0.88	0.07
31	14.31	1.69	0.84	12.65	1.46	0.85	0.08
32	13.86	1.75	0.81	12.26	1.51	0.82	0.08
33	13.44	1.80	0.78	11.89	1.55	0.80	0.08
34	13.04	1.86	0.76	11.54	1.60	0.78	0.08
35	12.67	1.91	0.74	11.21	1.65	0.75	0.09
36	12.32	1.96	0.72	10.90	1.70	0.73	0.09
37	11.99	2.02	0.70	10.60	1.74	0.71	0.09
38	11.67	2.07	0.68	10.32	1.79	0.69	0.09
39	11.37	2.13	0.66	10.06	1.84	0.67	0.10

e above values based on a maximum fiber-stress of 13 000 lb per sq in, rivet-hole th flanges deducted. See paragraph on Riveted Single-Beam and Double-Bean page 603, and the foot-note for same regarding fiber-stresses. Weights of girder ond to lengths, center to center of bearings.

Beams Supporting Brick Walls. In calculating the size of a girder to s port a brick wall, the structure of the wall should be carefully considered. the wall is without openings and does not support floor-beams, only that p of the wall included within the dotted lines, Fig. 10, need be considered as b supported by the girder. The beams in that case, however, should be made very STIFF, so as to have little DEFLECTION. If there are several openings above the girder, and especially if there is a pier over the middle part of it, as shown in Fig. 11, then the manner in which the loading is distributed should be carefully considered. In a case of this kind, only the dead weight included between the dotted lines AA and BB should be considered

Fig. 10. Triangular Loading of Beams under Brick Walls

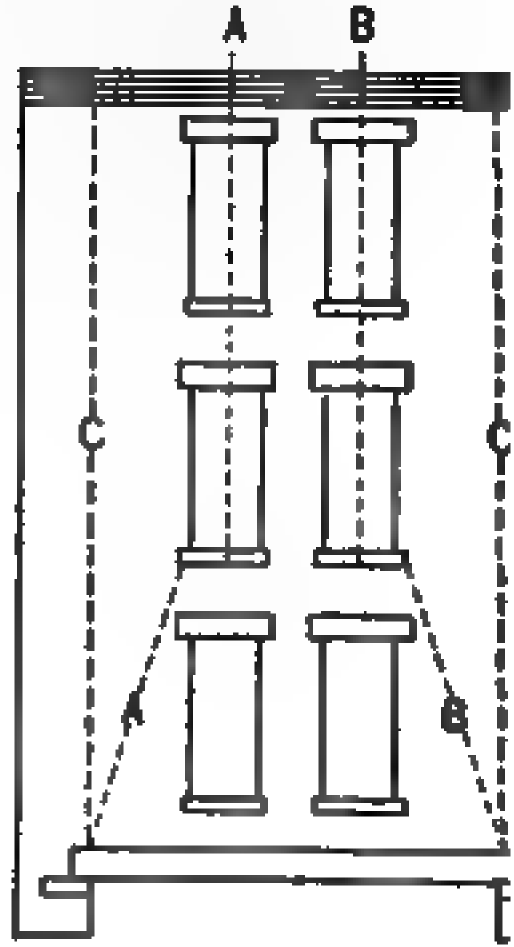


Fig. 11. Loading of Beams u Walls with Openings

as coming upon the girder, and proper allowance made for the CONCENTRA of the greater part of the load at or near the middle. If, however, the b windows are two-thirds their total width, or more, above the girder, th is more reasonable to suppose that the wall included between the line rests upon the girder, and also to consider that this load is UNIFORMLY TRIBUTED over it. When beams extend under the entire length of a which is more than 16 or 18 ft long, the weight of the entire wall rather the weight of a triangular part of it should be taken as coming upon beams; for, if they should bend, the wall would settle, and might push the supports and cause the whole structure to fall. (See, also, page 318.

5. Framing and Connecting Steel Beams and Girders

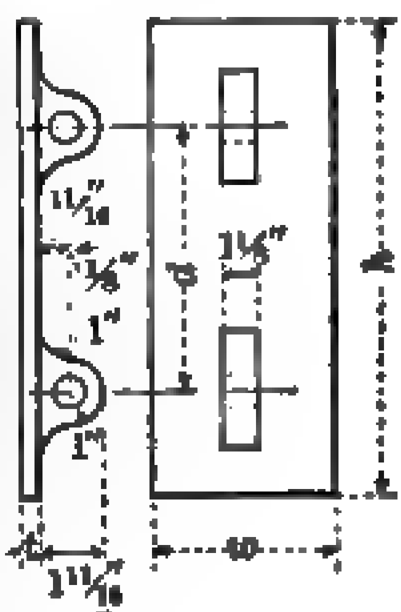
Standard Separators. When beams are used to support walls, or as g to carry floor-beams, they are often placed side by side; and should in cases be connected by means of BOLTS and cast-iron SEPARATORS f closely between the flanges of the beams. The office of these separators i measure, to hold in position the compression-flanges of the beams by preve SIDE DEFLECTION or BUCKLING, and also to unite the beams so as to them to act in unison as regards VERTICAL DEFLECTION. Separators s be provided at the supports, at points where heavy concentrated loa imposed, and at regular intervals of from 5 to 6 ft between. The ill tions, dimensions, etc., given in Table XVI, are for the STANDARD SEPAR in common use.

Table XVI.* Separators for Steel Beams

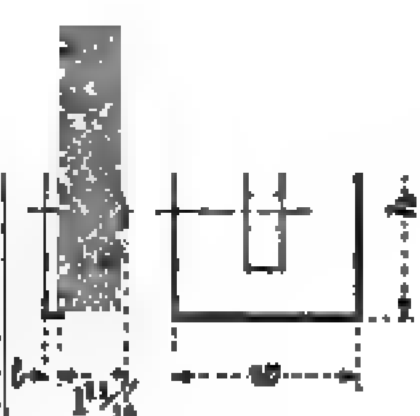
AMERICAN BRIDGE COMPANY STANDARD

Beams			Separators						¾-in bolts		
Weight per foot, lb	Center to center of beams, in	Out to out of flanges, in	Dimensions w in	h in	d in	t in	Weight, lb	Increase in weight for 1" add. width			Increase in weight for 1" add. length
85,110,105.1	8¾	16¾	8	20	12	⅜	31	3.6	10¼	3.4	0.25
100	8	15½	7¼	20	12	⅜	28	3.6	10	3.2	0.25
95 and 90	8	15¼	7¼	20	12	⅜	28	3.6	10	3.2	0.25
85	8	15¼	7½	20	12	⅜	29	3.6	9½	3.1	0.25
79.9	8	15	7½	20	12	⅜	29	3.6	9½	3.1	0.25
100 and 95	8	15¼	7	16	12	⅜	22	2.9	10	3.2	0.25
90	7½	14¾	6¾	16	12	⅜	22	2.9	9½	3.1	0.25
85 and 81.4	7½	14½	6¾	16	12	⅜	22	2.9	9	3.0	0.25
75	7½	14	6¾	16	12	⅜	22	2.9	9	3.0	0.25
70	7	13½	6½	16	12	⅜	21	2.9	9	3.0	0.25
65.4	7	13¼	6½	16	12	⅜	21	2.9	8½	3.0	0.25
90	8	15¼	7	14	9	⅜	20	2.5	10	3.2	0.25
85 and 80	8	15¼	7¼	14	9	⅜	21	2.5	10	3.2	0.25
75.6	8	15	7½	14	9	⅜	21	2.5	10	3.2	0.25
70 and 65	7	13¼	6¼	14	9	⅜	18	2.5	9	3.0	0.25
60	7	13¼	6½	14	9	⅜	19	2.5	8½	3.0	0.25
54.7	7	13	6½	14	9	⅜	19	2.5	8½	3.0	0.25
75	7	13¼	6	11	7½	½	12	1.6	9	3.0	0.25
70 and 65	7	13½	6¼	11	7½	½	12	1.6	9	3.0	0.25
60.8	6½	12½	5¾	11	7½	½	11	1.6	8	2.7	0.25
55	6½	12¼	5¾	11	7½	½	11	1.6	8	2.7	0.25
50 and 45	6½	12¼	6	11	7½	½	12	1.6	8	2.7	0.25
42.9	6½	12	6	11	7½	½	12	1.6	8	2.	
55	6	11¾	5¾	8¾	5	½	9	1.3	8	2.	
50	6	11½	5½	8¾	5	½	9	1.3	8	2.	
45	6	11¼	5¼	8¾	5	½	9	1.3	7½	2.	
40 and 35	6	11¼	5¼	8¾	5	½	9	1.3	7½	2.	
31.8	6	11	5¼	8¾	5	½	9	1.3	7½	2.	
40	5¼	10¾	4¾	7¼	....	½	6	1.1	7½	1.	
35	5½	10½	4¾	7½	....	½	6	1.1	7	1.	
30	5¼	10¼	5	7½	....	½	7	1.1	7	1.	
25.4	5¼	10	5	7½	....	½	7	1.1	7	1	
35	5	10	4½	6½	....	½	5	0.9	7	1.	
30	5	9¾	4½	6½	....	½	5	0.9	6½	1.	
25	5	9½	4½	6½	....	½	5	0.9	6½	1	
21	5	9¼	4½	6½	....	½	5	0.9	6½	1.2	0.13
25.5	4½	9	4	5½	....	½	4	0.8	6	1.1	0.13
23	4½	8¾	4	5½	....	½	4	0.8	6	1.1	0.13
20 and 18.4	4½	8½	4	5½	....	½	4	0.8	6	1.1	0.13
20	4½	8½	4	5	....	½	4	0.7	6	1.1	0.13
17.5	4¼	8¼	4	5	....	½	4	0.7	6	1.1	0.13
15.3	4½	8¼	4¼	5	....	½	4	0.7	6	1.1	0.13
17.25	4	7¾	3½	4½	....	½	4	0.6	5½	1.1	0.13
14.75	4	7½	3½	4½	....	½	4	0.6	5½	1.1	0.13
12.5	4	7½	3¼	4½	....	½	4	0.6	5½	1.1	0.13

Diagrams



¾" cored holes



¾" cored hole

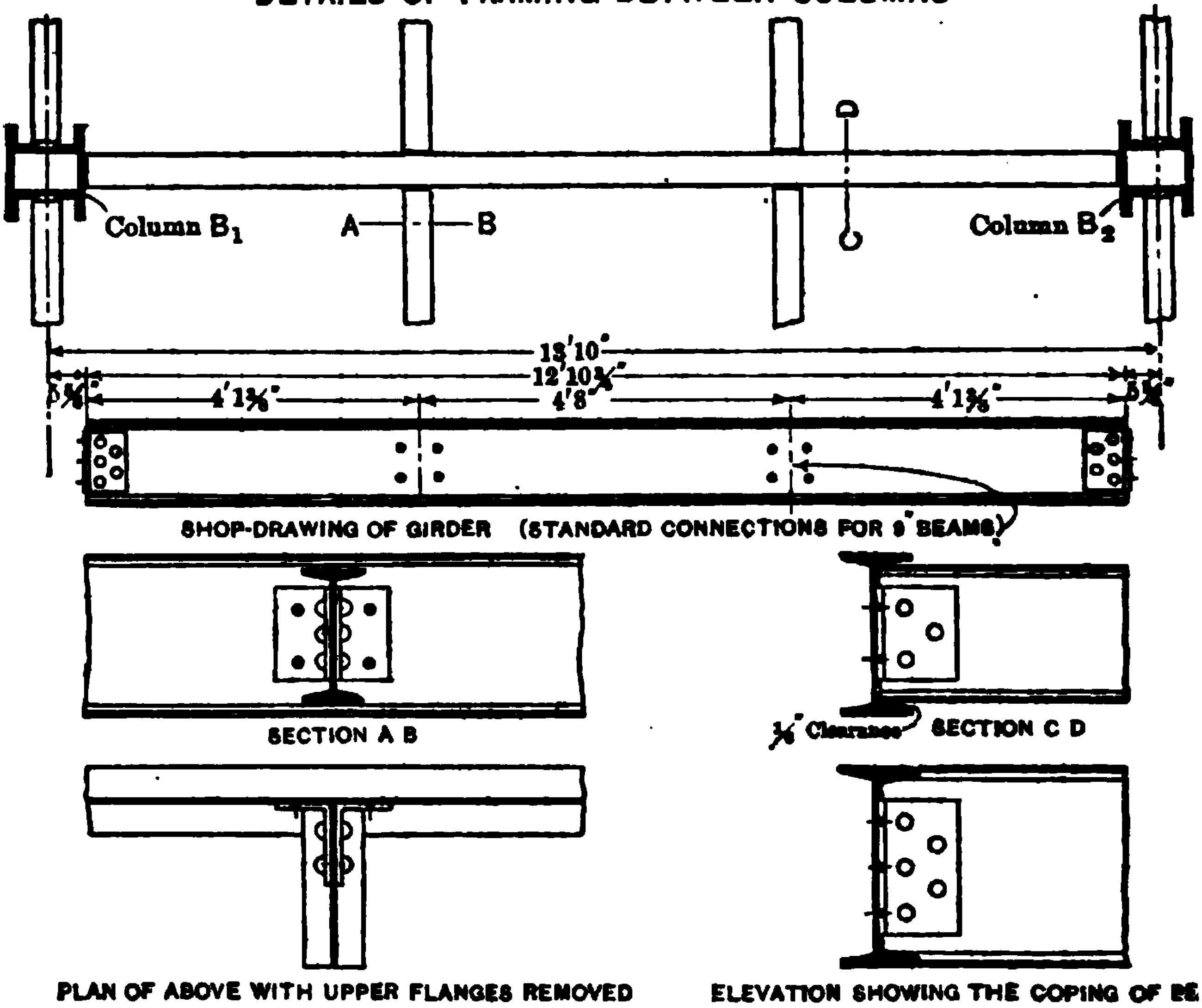
For 5-in, 4-in and 3-in beams, use 1-in gas-pipes, 3¼, 3 and 2¾-in long, respectively

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Gas-Pipe Separators. Separators formed of pieces of GAS-PIPE, cut to desired lengths and slipped over the bolts are often used by contractors. (bottom of Table XVI.) Such separators permit the beams to act independe of each other, and should not be used in any place where one beam is liabl receive a greater load than the other; and as this condition exists in alm every case where two or more beams are used together, it follows that "c iron separators, made to fit the space between the beams," should be speci in almost every instance. As noted in Table XVI, gas-pipe may sometime used for 5, 4 and 3-in beams. Separators with two bolts should be used beams 12 in or more in depth. For 12-in beams one bolt is sometimes used w the load is light; for beams under 12 in in depth one bolt is sufficient.

FRAMING

DETAILS OF FRAMING BETWEEN COLUMNS



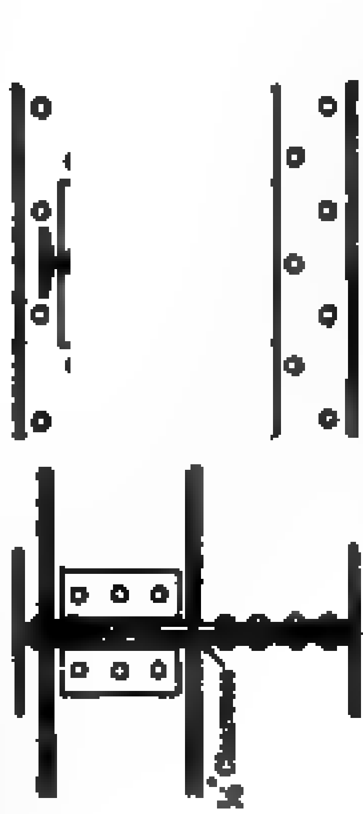
CONNECTIONS FOR BEAMS AND GIRDERS

"Connection-angles shall in no case be less in thickness than the web of the b or girder to which they are fastened, nor shall the width be less than ⅓ the dept the beam, except that no angle-knee shall be less than 2½" wide nor required t more than 6" wide. Web-angles, the full depth of the web, must be used for all gi connections."

Fig. 12. Framing of Steel I Beams and Girders

Beam-Connections. Steel beams and channels are FRAMED together means of short pieces of angles, which are usually riveted to the floor-bea tail-beam and bolted to the girder. The angles are always used in pairs, on each side of the beam. If the floor-beam is framed flush, either with the to bottom of the girder, or if two beams of the same height are framed together, end of the beam supported should be COPED, or cut to fit the shape of the gi or supporting beam. The maximum clearance-space allowed between the

ab with that of the beam girder and coped to fit it, except where the girder is on a
ast 1¼ in below the beams so the latter may be ... Smaller beams
and coped. All openings in the floors, except ... the full width
all sides. Built-up girders are to be set with ... more than 5 in
necessary and required 3½ by 3½ by ¼-in ... riveted to the
roof and vault-arching.



(b) BEAM-CONNECTION OF ANGLE-KNEES

d channels between girders and not more than ¼ at each
be true to the drawings and an error of more than 1/16 is
variation of more than ½ in in the length of beams suf-
ufficient cause for rejection."

Beam
line wit
are to b
of the
above t
beams.

g to Riveted Plate Girder

m 1/16 in in the smaller beams to ¼ in in the
w various details of beams framed together
beam rests on top of another beam or girder,
secured by means of a pair of wrought-iron

CLIPS, shown in Fig. 14, shaped so as to fit closely the top flange of the gird and either bolted or riveted to the opposite sides of the lower flange of the flo beam.

Fig. 16 shows one method of framing the ends of wooden floor-joists to st beams, a 4 by 3 by ⅜-in angle being riveted the whole length of the steel bea by ¾-in rivets, about 6 in apart. The joists are usually secured by iron

Fig. 14. Clip for Fastening Steel Beam on Top of Another

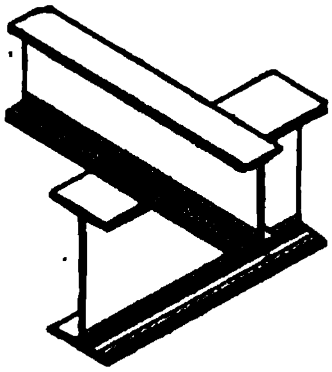


Fig. 15. Steel Beams Fastened One on the Other by Clips

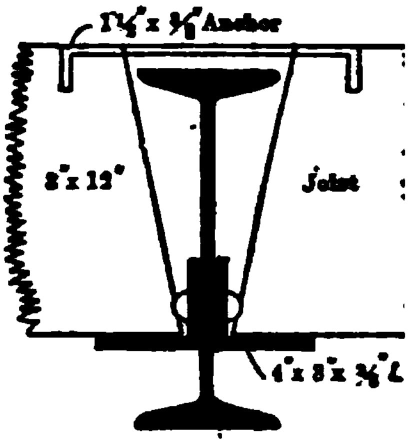


Fig. 16. Framing of Wood Joists to Steel I Beam

CLAMPS or ANCHORS, and framed about 1 in above the upper flange of the bea to allow for settlement. If these joists are over 3 ft apart, short lengths of ang may be placed under each one.*

Standard Connection-Angles for I Beams and Channels. The size the angles and the number of rivets used for connecting steel beams, va somewhat with different shops and with different structural engineers, so th there cannot be said to be a universal standard. The variations in the diffe ent STANDARDS, however, are not very great, and as the connections adopt by the Carnegie Steel Company are perhaps the most used, the author has lected them for illustration in Table XVII. The CONNECTIONS have been p portioned with a view to covering most cases occurring in ordinary practi with the usual relations of depth of beam to length of span. In extreme stances, however, where beams of short relative span-lengths are loaded to th full capacity, or when beams frame opposite each other into another beam wi web-thickness less than $\frac{5}{16}$ in, it may be found necessary to make provision additional strength in the connections. The LIMITING SPAN-LENGTHS, al at and above which the standard connection-angles may be used with perfe safety, are also given in Table XVIII.

* For details of the framing of floor-beams and girders, see Chapters XXI and XX and also Professor Nolan's revised Chapters II and VII of Kidder's Building-Constructi and Superintendence, Part II, Carpenters' Work.

Table XVII.* Connections for Steel Beams

AMERICAN BRIDGE COMPANY STANDARD

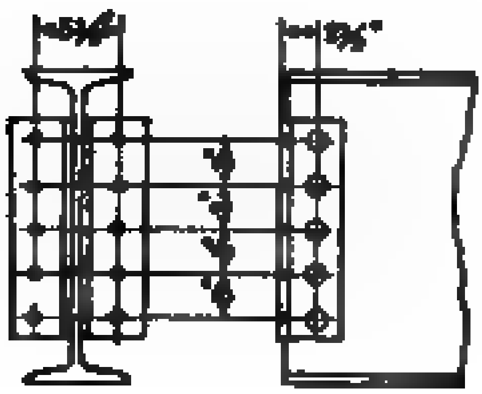
27"

2 Ls 4"x 4"x ½"x 1' 8½"
Weight 46 lb

24"

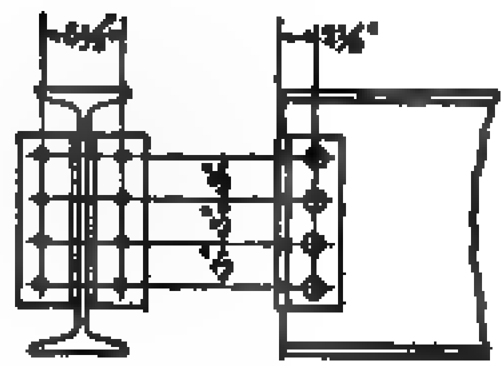
2 Ls 4"x 4"x ½"x 1' 5½"
Weight 39 lb

21"



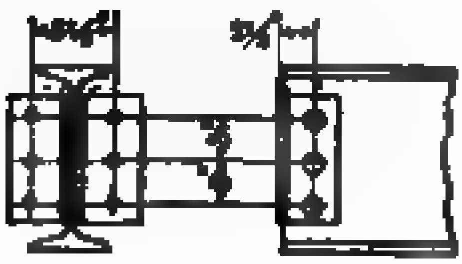
2 Ls 4"x 4"x ½"x 1' 2½"
Weight 33 lb

20", 18", 15"



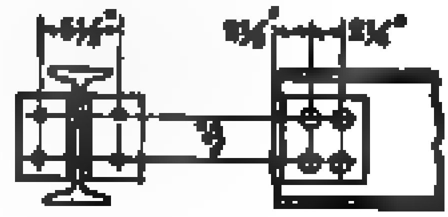
2 Ls 4"x 4"x 7/16"x 0' 11½"
Weight 23 lb

12"



2 Ls 4"x 4"x 7/16"x 0' 8½"
Weight 17 lb

10", 9", 8"

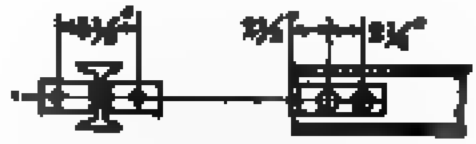


2 Ls 6"x 4"x ⅜"x 0' 5½"
Weight 13 lb

7", 6", 5"

2 Ls 6"x 4"x ⅜"x 0' 3"
Weight 7 lb

4", 3"



2 Ls 6"x 4"x ⅜"x 0' 2"
Weight 5 lb

Rivets and bolts ¾" diameter
Weights given are for ¾" shop rivets and angle-connections; about 10 per cent should be added for field-rivets or bolts

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

Table XVIII.* Limiting Values of Connections for Steel Beams

I beams		Value of web-connection	Values of outstanding legs of connection-angles				
			Field-rivets			Field-bolts	
Depth, in	Weight, lb per ft	Shop-rivets in enclosed bearing, lb	¾-in rivets or turned bolts, single shear, lb	Min. allowable span, uniform load, ft	*t*, in	¾-in rough bolts, single shear, lb	Min. allowable span, uniform load, ft
27	90	82 530	61 900	18.9	5/8	49 500	23.6
24	79.9	67 500	53 000	17.5	5/8	42 400	21.9
	74.2	64 260	53 000	16.4	5/8	42 400	20.4
21	60.4	48 150	44 200	14.2	5/8	35 300	17.8
20	65.4	45 000	35 300	17.6	5/8	28 300	22 1
18	54.7	41 400	35 300	13.3	5/8	28 300	16.7
	48.2	34 200	35 300	12.8	9/16	28 300	15.4
15	42.9	36 900	35 300	8.9	5/8	28 300	11.1
	37.3	29 880	35 300	9.7	1/2	28 300	10.2
12	31.8	23 600	26 500	8.1	9/16	21 200	9.0
	27.9	19 170	26 500	9.2	7/16	21 200	9.2
10	25.4	27 900	17 700	7.4	5/8	14 100	9.2
	22.4	22 680	17 700	6.8	5/8	14 100	8.6
9	21.8	26 100	17 700	5.7	5/8	14 100	7.1
8	18.4	24 300	17 700	4.3	5/8	14 100	5.4
	17.5	18 900	17 700	4.4	5/8	14 100	5.5
7	15.3	11 300	8 800	6.2	5/8	7 100	7.8
6	12.5	10 400	8 800	4.4	5/8	7 100	5.5
5	10.0	9 500	8 800	2.9	5/8	7 100	3 6
4	7.7	8 600	8 800	2.2	9/16	7 100	2.7
3	5.7	7.700	8 800	1.3	1/2	7 100	1.4

Allowable Unit Stress in Pounds per Square Inch †

Single shear	Rivets............shop	12 000	Bearing	Rivets, enclosed..shop	30
	Rivets and turned bolts............field	10 000		Rivets, one side..shop	24
	Rough bolts.......field	8 000		Rivets and turned bolts............field	20
				Rough bolts......field	16

t = Web-thickness, in bearing, to develop maximum allowable reactions, wh beams frame opposite

Connections are figured for bearing and shear (no moment considered)

The above values agree with tests made on beams under ordinary conditions use

Where the web is enclosed between connection-angles (enclosed bearing), valu are greater because of the increased efficiency due to friction and grip

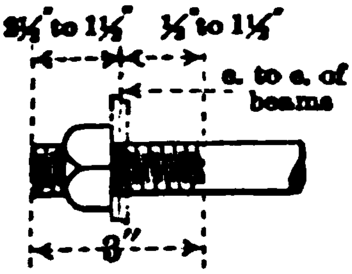
Special connections must be used when any of the limiting conditions giv above are exceeded, as when an end-reaction from a loaded beam is greater th the value of the connection of the shorter span with the beam fully loaded; or a l thickness of web when maximum allowable reactions are used

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

† For slight variations from these values, see Chapter XXVIII, Table I and Ch XXX, 4, Stresses.

Table XIX.* Lengths and Weights of Tie-Rods and Anchors for Steel Beams

AMERICAN BRIDGE COMPANY STANDARD



¾-INCH TIE-RODS

LENGTHS AND WEIGHTS FOR VARIOUS DISTANCES CENTER TO CENTER OF BEAMS

Weights include two nuts

C to C	L'th	Wgt	C to C	L'th	Wgt	C to C	L'th	Wgt	C to C	L'th	Wgt
ft in	ft in	lb	ft in	ft in	lb	ft in	ft in	lb	ft in	ft in	lb
1 0	1 3	2.30	1 3	1 6	2.67	1 6	1 9	3.05	1 9	2 0	3.42
2 0	2 3	3.80	2 3	2 6	4.17	2 6	2 9	4.55	2 9	3 0	4.92
3 0	3 3	5.30	3 3	3 6	5.67	3 6	3 9	6.05	3 9	4 0	6.42
4 0	4 3	6.80	4 3	4 6	7.17	4 6	4 9	7.55	4 9	5 0	7.92
5 0	5 3	8.30	5 3	5 6	8.67	5 6	5 9	9.05	5 9	6 0	9.42
6 0	6 3	9.80	6 3	6 6	10.17	6 6	6 9	10.55	6 9	7 0	10.92
7 0	7 3	11.30	7 3	7 6	11.67	7 6	7 9	12.05	7 9	8 0	12.42
8 0	8 3	12.80	8 3	8 6	13.17	8 6	8 9	13.55	8 9	9 0	13.92

For strength of rods, see Table II, page 388.

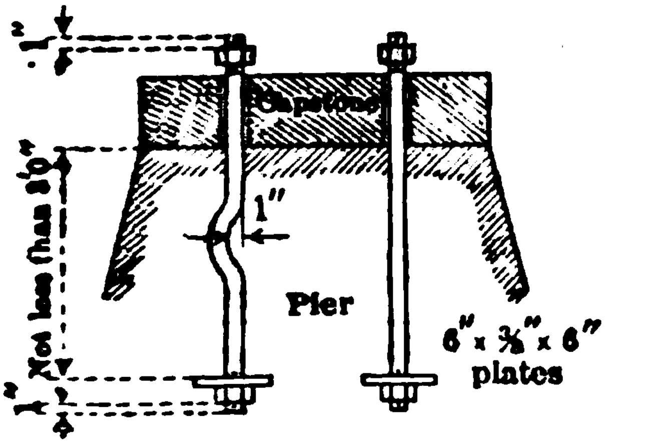
Anchors *

SWEDGE-BOLT

Weight includes nut

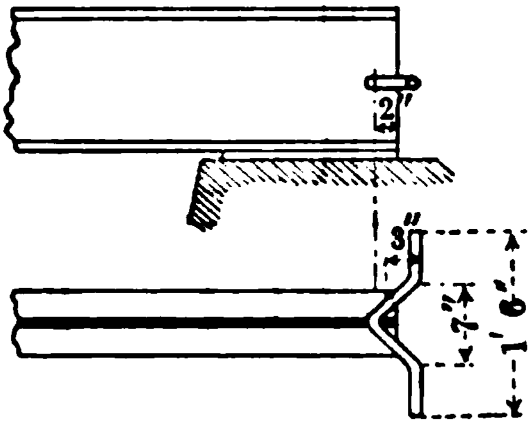
BUILT-IN ANCHOR-BOLTS

Diameter	Length	Weight
in	ft in	lb
¾	0 9	1.3
⅞	1 0	2.3
1	1 0	3.1
1¼	1 8	6.1



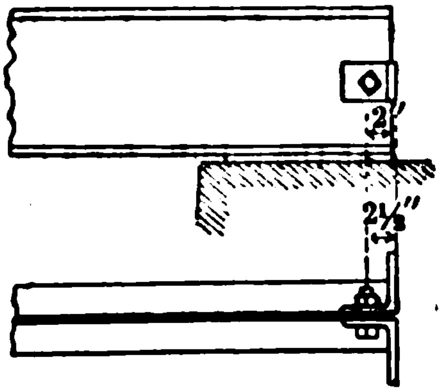
When center to center of anchors is less than width of washer, use washer with two holes

GOVERNMENT ANCHOR



¾-in rod, 1 ft 9 in long. Wt., 3 lb

ANGLE-ANCHOR



Two angles, 6 by 4 by 7/16 by 2½ in
Weight with ¾-in bolts, 7 lb

For bearing-plates, bases, etc., see Chapter XIII.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

CHAPTER XVI

STRENGTH OF CAST-IRON LINTELS AND WOODE BEAMS

By

F. H. KINDL

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1. Cast-Iron Lintels

Form of Cross-Section. Owing to the fact that the resistance of cast i to tension is only about one-fifth of its resistance to compression, the shapes beams most economical for wrought iron or steel would be wasteful for c iron. The extreme brittleness of cast iron, and the danger of flaws in castin render it an undesirable material for resisting transverse stress. About t only form in which cast-iron beams are now used in building-construction this country is in the shape of LINTELS for supporting brick or stone walls, places where a flat soffit is desired, and the walls a not to be plastered. CAST-IRON LINTELS are a occasionally used over store-fronts, the face of t lintel being paneled and molded for architectu effect.

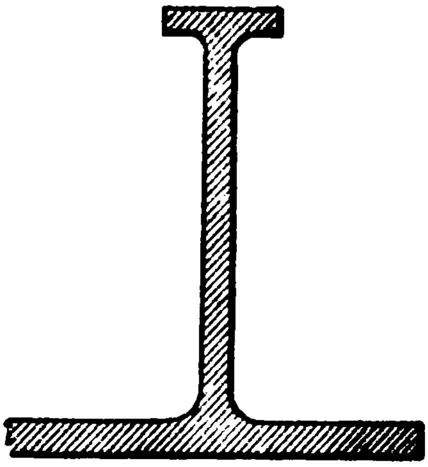


Fig. 1. Cross-section of Cast-iron Lintel of Ideal Form

Experiments on Cast-Iron Beams. Bef wrought-iron I beams were manufactured, CA IRON BEAMS were frequently used as the o available ones, other than those of wood or sto Early in the nineteenth century Eaton Hodgk son, an English engineer, made a series of exp ments with cast-iron beams, from which he fou that the form of cross-section of a beam of t material which will resist the greatest transve stress is that shown in Fig. 1, in which there is six times more metal in bottom than in the top flange. The relative thicknesses of the three parts, web, the top flange and the bottom flange, may be, with advantage, as 5, 6 a 8, respectively.

Strength of Cast-Iron Beams. If made with these proportions, the wi of the top flange will be equal to one-third that of the bottom flange. As result of his experiments, Hodgkinson gave the following rule for the breaki weight at the middle for a cast-iron beam of this form:

$$\text{Breaking-load in tons} = \frac{\left(\begin{array}{c}\text{area of bottom flange}\\ \text{in square inches}\end{array}\right) \times \left(\begin{array}{c}\text{depth}\\ \text{in inches}\end{array}\right) \times 2.426}{\text{clear span in feet}}$$

This rule, although largely empirical, agreed very well with the few exp ments that were made. Structural engineers, however, use the general form las for the strength of beams, as given in Chapter XV, except that the SECTI MODULUS is found by dividing the MOMENT OF INERTIA by the distance of neutral axis from the bottom of the beam, and the SAFE TENSILE STRENGT

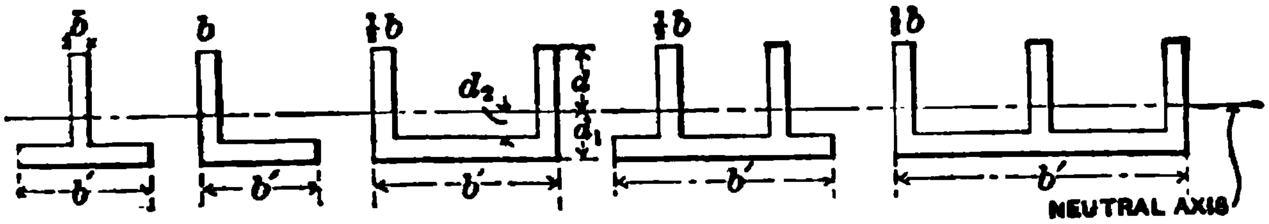
used in the FLEXURE-FORMULA. Thus the general formula for a beam supported at both ends and with the load uniformly distributed, as given in Chapter XV, page 560, is:

Safe load in pounds $= \frac{2}{3}\,\frac{I/c}{l} \times S_t$. As S_t, the safe tensile strength for cast iron should be taken at 3 000 lb, this formula becomes

$$\text{Safe load in pounds} = \frac{2\,000\,I/c}{l} \qquad (2)$$

and, for either section given below,

$$I/c = \frac{\text{Moment of inertia}}{d_1}$$



The MOMENT OF INERTIA is computed by the formula (see page 337)

$$I = \frac{bd^3 + b'd_1^3 - (b' - b)d_2^3}{3} \qquad (3)$$

in which b denotes the combined thickness of the webs, and the distances d, d_1 and d_2 are measured from the NEUTRAL AXIS, which must pass through the CENTER OF GRAVITY of the section. The center of gravity may be found by the method explained in Chapter VI. This formula may be used for any of the above sections when the depth does not exceed the width, and the thickness of each web is at least equal to the thickness of the flange. In lintels with a single web it is well to make the thickness of the web ¼ or ⅛ in greater than the thickness of the flange. For a beam with a cross-section like that shown in Fig. 1, Formula (2) agrees very closely with Formula (1), when a factor of safety of six is used.

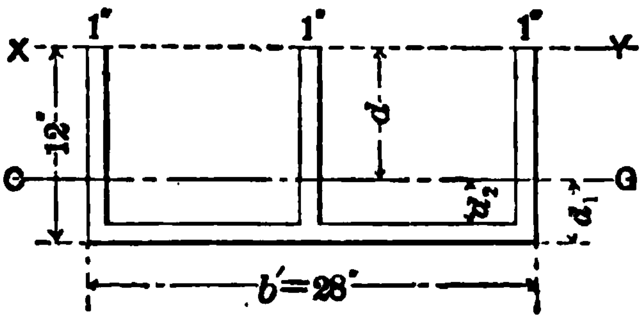


Fig. 2. Cross-section of Cast-iron Lintel with Three Webs

Example. The following example illustrates the application of Formula (3): It is required to compute the safe load for a cast-iron lintel having the section shown in Fig. 2 and a clear span of 10 ft. The load is uniformly distributed, and the thickness of the metal 1 in.

Solution. The first step is the finding of distance d, that the center of gravity, through which the neutral axis of the cross-section passes, is below the top surface of the beam. This is found by taking the moments of the areas of the cross-sections of webs and flange about the line XY, and dividing their sum by the area of the entire section. (See page 294.) Each web-section is 11 in long and 1 in thick; hence the area of each is 11 sq in. The MOMENTS OF THE THREE WEBS about XY will then be $3 \times 11 \times 5\frac{1}{2} = 181.5$

the MOMENT OF THE FLANGE about $XY = 28 \times 11\frac{1}{2} = 322$

$\quad 503.5$

The area of the entire cross-section = 61 sq in

$503.5 \div 61 = 8.25 = d$ in

Then	$d = 8.25$ in	$d^3 = 561.5$	$b = 3$ in
	$d_1 = 3.75$ in	$d_1^3 = 52.7$	$b' = 28$ in
	$d_2 = 2.75$ in	$d_2^3 = 20.8$	

The MOMENT OF INERTIA is next found by Formula (3):

$$I = \frac{3 \times 561.5 + 28 \times 52.7 - 25 \times 20.8}{3} = 880$$

$I/c = 880/3.75$. From Formula (2) the safe load = $(2\,000 \times 234.6)/10 = 46\,9$ lb, or 23.4 tons.

Ends and Brackets of Cast-Iron Lintels. When a lintel, the cross-tion of which has the shape of an inverted T (⊥), is used over a single opening,

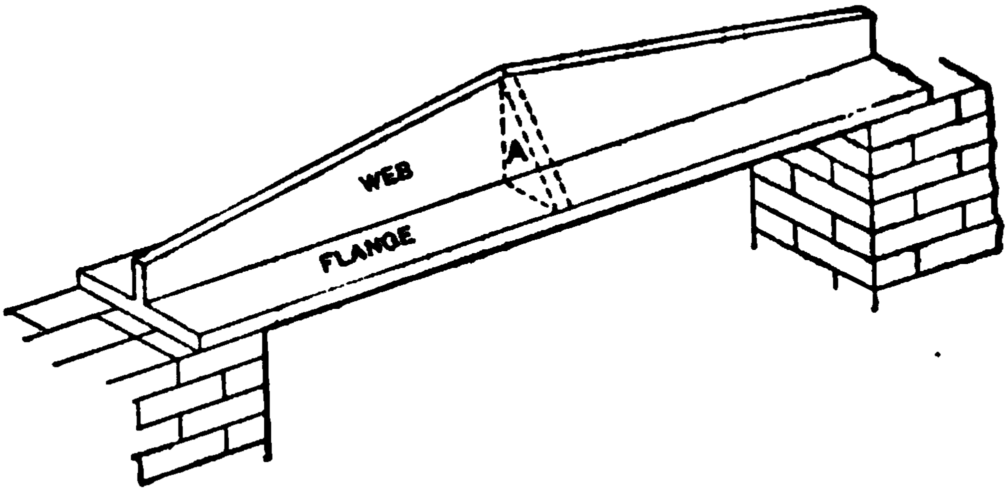


Fig. 3. Cast-iron Lintel with Tapering Web

web may be tapered towards the ends, as in Fig. 3, without affecting the streng If the flange is more than 8 in wide, brackets should be cast in the middle, a A, Fig. 3.

When CONTINUOUS LINTELS are used over store-fronts or similar pla ends should be cast on the lintels, as in Fig. 4, and the ends of abutting lin

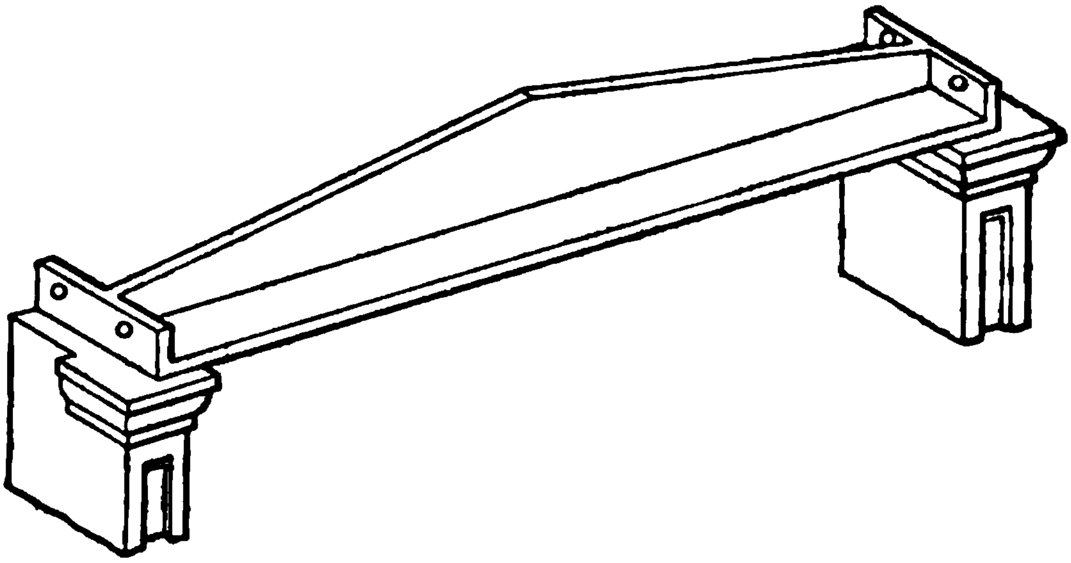


Fig. 4. Cast-iron Lintel with Ends for Bolting

bolted together. All lintels with two or three webs should have solid ends necting the webs.

Tables of Strength of Cast-Iron Lintels. The tables on the follo pages have been computed in accordance with Formula (2). The weight of

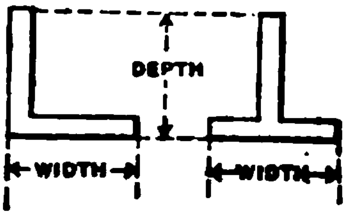
lintel itself should be deducted from the safe load. In using these tables it should be remembered that the values are for loads UNIFORMLY DISTRIBUTED. If the load is CONCENTRATED AT THE MIDDLE, it should be multiplied by 2. If at some other point than the middle, the load should be multiplied by the value given on pages 566 and 632, which most nearly corresponds with the position of the load. For other spans than those given, the distributed load should be multiplied by the span, and the lintel used which has a COEFFICIENT OF STRENGTH C (Table I) just above the product thus obtained. (For explanation of coefficient of strength, see Chapter XV, page 556.)

Example. It is required to support a 12-in brick wall, 10 ft high, over an opening 5 ft 6 in wide, with a cast-iron lintel. At a distance of 22 in from one support, a girder, which may bring a load of 9 600 lb on the lintel, enters the wall. What should be the dimensions of the lintel?

Solution. At 110 lb per cu ft, the wall above the lintel weighs $10 \times 5\frac{1}{2} \times 110 = 6\,050$ lb. As 22 in is one-third of the span, the concentrated load is multiplied by 1.78 (page 632), making the load 17 088 lb. The total equivalent distributed load is then 23 138 lb. Multiplying this by the span there results 127 259 lb, or 63.6 tons, as the least value for the coefficient of strength *C*. From the table, it is found that a 12 by 10-in lintel, 1 in thick, with one web, has a coefficient of strength of 72.2; and that a 12 by 8 by $1\frac{1}{4}$-in lintel with two webs, has a coefficient of strength of 69.9. A lintel with two webs is best for a 12-in wall, and interpolating between the values of *C* for the 1-in and 2-in thicknesses of the 12 by 8-in lintel, 65.4 is found to be the value of *C* for a thickness of $1\frac{1}{8}$ in. This exceeds the required value by enough to more than compensate for the weight of the lintel itself; hence a 12 by 8 by $1\frac{1}{8}$-in lintel with two webs is used.

Flaws in Castings. Owing to the liability of flaws in the castings, cast-iron lintels should always be carefully inspected before being accepted.

Table I. Safe Distributed Loads in Tons for Cast-Iron Lintels

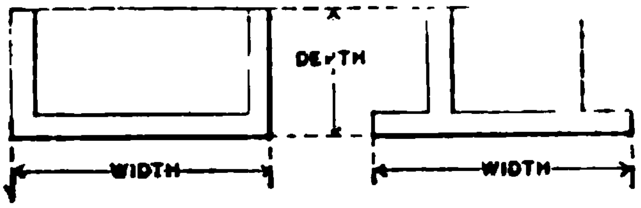
LINTELS OF  SHAPES

Loads include weights of lintels. Maximum tensile stress 3 000 lb per sq in. remarks, pages 622 and 623.

Size, width by depth, in	Thickness of metal, in	Weight per foot, lb	C, tons	Span in feet							
				5	6	7	8	9	10	11	1
6× 6	3/4	26.3	15.9	3.18	2.65	2.27	1.98	1.76	1.59	1.44	1.
	1	34.4	19.0	3.80	3.16	2.71	2.37	2.11	1.90	1.72	1.
	1 1/4	42.0	21.5	4.30	3.58	3.07	2.68	2.39	2.15	1.95	1.
7× 6	3/4	28.6	17.8	3.56	2.96	2.54	2.22	1.98	1.78	1.61	1.
	1	37.5	21.3	4.26	3.55	3.04	2.66	2.36	2.13	1.93	1.
	1 1/4	45.9	24.0	4.80	4.00	3.43	3.00	2.66	2.40	2.18	2.
7× 7	3/4	31.0	22.6	4.52	3.76	3.23	2.82	2.51	2.26	2.05	1.
	1	40.6	27.5	5.50	4.58	3.93	3.43	3.05	2.75	2.50	2.
	1 1/4	49.8	31.4	6.28	5.23	4.49	3.92	3.49	3.14	2.85	2.
8× 6	3/4	31.0	19.6	3.92	3.26	2.80	2.45	2.18	1.96	1.78	1.
	1	40.6	23.4	4.68	3.90	3.34	2.92	2.60	2.34	2.12	1.
	1 1/4	49.8	26.4	5.28	4.40	3.77	3.30	2.93	2.64	2.40	2.
8× 7	3/4	33.3	25.0	5.00	4.16	3.57	3.12	2.77	2.50	2.27	2.
	1	43.7	30.3	6.06	5.05	4.33	3.79	3.36	3.03	2.75	2.
	1 1/4	53.7	34.8	6.96	5.80	4.97	4.35	3.86	3.48	3.16	2.
8× 8	3/4	35.6	30.6	6.12	5.10	4.37	3.82	3.40	3.06	2.78	2.
	1	46.8	37.6	7.52	6.26	5.37	4.70	4.18	3.76	3.41	3.
	1 1/4	57.6	43.4	8.68	7.23	6.20	5.42	4.82	4.34	3.94	3.
8× 9	3/4	38.0	36.5	7.30	6.08	5.21	4.56	4.05	3.65	3.31	3.
	1	50.0	45.2	9.04	7.53	6.45	5.65	5.02	4.52	4.11	3.
	1 1/4	61.5	52.6	10.52	8.76	7.51	6.57	5.84	5.26	4.78	4
12× 6	3/4	40.4	26.5	5.30	4.41	3.78	3.31	2.94	2.65	2.41	2.
	1	53.1	31.6	6.32	5.26	4.51	3.95	3.51	3.16	2.87	2.
	1 1/4	65.4	34.8	6.96	5.80	4.97	4.35	3.86	3.48	3.16	2
12× 8	3/4	45.0	41.7	8.34	6.95	5.95	5.21	4.63	4.17	3.79	3
	1	59.4	51.2	10.24	8.53	7.31	6.40	5.69	5.12	4.65	4
	1 1/4	73.2	58.5	11.70	9.75	8.35	7.31	6.50	5.85	5.32	4
12×10	3/4	49.8	58.0	11 60	9.66	8.28	7.25	6.44	5.80	5.27	4
	1	65.6	72.2	14.44	12.03	10.31	9.02	8.02	7.22	6.56	6
	1 1/4	81.0	83.8	16.76	13.96	11.97	10.47	9.31	8.38	7.62	6
12×12	3/4	54.4	75.2	15.04	12.53	10.74	9.40	8.35	7.52	6.83	6
	1	71.9	94.8	18.96	15.80	13.54	11.85	10.53	9.48	8.62	7
	1 1/4	88.9	111.5	22.30	18.58	15.92	13.93	12.39	11.15	10.12	9

Table 1 (Continued). Safe Distributed Loads in Tons for Cast-Iron Lintels

LINTELS OF

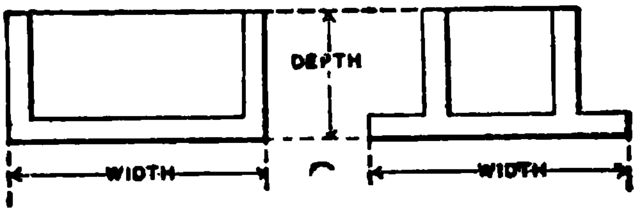


SHAPES

Loads include weights of lintels. Maximum tensile stress 3 000 lb per sq in. See remarks, pages 622 and 623.

Size, width by depth, in	Thickness of metal, in	Weight per foot, lb	C, tons	Span in feet							
				5	6	7	8	9	10	11	12
	¾	52.7	31.7	6.34	5.28	4.53	3.96	3.52	3.17	2.88	2.64
x 6	1	68.8	37.6	7.52	6.26	5.37	4.70	4.18	3.76	3.42	3.13
	1¼	84.0	43.0	8.60	7.16	6.14	5.37	4.77	4.30	3.91	3.58
	¾	62.1	49.5	9.90	8.25	7.07	6.19	5.50	4.95	4.50	4.12
x 8	1	81.3	60.9	12.18	10.15	8.70	7.61	6.76	6.09	5.53	5.07
	1¼	99.6	69.9	13.98	11.65	9.98	8.73	7.76	6.99	6.35	5.82
	¾	57.4	35.5	7.10	5.91	5.07	4.43	3.94	3.55	3.22	2.96
x 6	1	75.0	42.0	8.40	7.00	6.00	5.25	4.66	4.20	3.82	3.50
	1¼	91.8	48.0	9.60	8.00	6.85	6.00	5.33	4.80	4.36	4.00
	¾	66.8	55.4	11.08	9.23	7.91	6.92	6.15	5.54	5.03	4.61
x 8	1	87.5	68.1	13.62	11.35	9.73	8.51	7.56	6.81	6.19	5.67
	1¼	107.4	78.8	15.76	13.13	11.25	9.85	8.75	7.88	7.16	6.56
	¾	62.1	39.1	7.82	6.51	5.58	4.88	4.34	3.91	3.55	3.25
x 6	1	81.3	46.8	9.36	7.80	6.68	5.85	5.20	4.68	4.25	3.90
	1¼	99.6	52.9	10.58	8.81	7.55	6.61	5.88	5.29	4.81	4.40
	¾	71.5	61.4	12.28	10.23	8.77	7.67	6.82	6.14	5.58	5.11
x 8	1	93.8	74.6	14.92	12.43	10.65	9.32	8.29	7.46	6.78	6.21
	1¼	115.2	86.8	17.36	14.46	12.40	10.85	9.64	8.68	7.89	7.23
	¾	71.5	47.2	9.44	7.86	6.74	5.90	5.24	4.72	4.29	3.93
x 6	1	93.8	55.1	11.02	9.18	7.87	6.88	6.12	5.51	5.01	4.59
	1¼	115.2	62.0	12.40	10.33	8.85	7.75	6.88	6.20	5.63	5.16
	¾	80.8	72.6	14.52	12.10	10.37	9.07	8.06	7.26	6.60	6.05
x 8	1	106.2	89.5	17.90	14.91	12.78	11.18	9.94	8.95	8.13	7.45
	1¼	130.8	102.5	20.50	17.08	14.64	12.81	11.39	10.25	9.31	8.54
	¾	90.2	100.5	20.10	16.75	14.35	12.56	11.16	10.05	9.13	8.37
x 10	1	118.8	125.4	25.08	20.90	17.91	15.67	13.93	12.54	11.40	10.45
	1¼	146.5	146.8	29.36	24.46	20.97	18.35	16.31	14.68	13.34	12.23
	¾	99.6	122.6	24.52	20.43	17.51	15.32	13.62	12.26	11.14	10.21
x 12	1	131.3	158.0	31.60	26.33	22.57	19.75	17.55	15.80	14.36	13.16
	1¼	162.1	189.5	37.90	31.58	27.07	23.68	21.05	18.95	17.22	15.79
	¾	90.2	83.4	16.68	13.90	11.91	10.42	9.26	8.34	7.58	6.95
x 8	1	118.8	102.4	20.48	17.06	14.63	12.80	11.37	10.24	9.31	8.53
	1¼	146.5	117.0	23.40	19.50	16.71	14.62	13.00	11.70	10.63	9.75

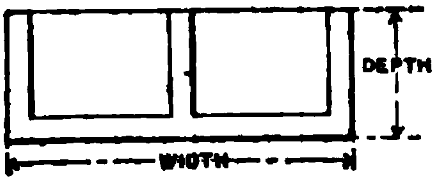
Table I (Continued). Safe Distributed Loads in Tons for Cast-Iron Lintels

LINTELS OF 

SHAPES

Loads include weights of lintels. Maximum tensile stress 3 000 lb per sq in. S
remarks, pages 622 and 623.

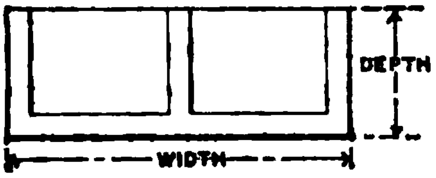
Size, width by depth, in	Thickness of metal, in	Weight per foot, lb	C, tons	Span in feet							
				5	6	7	8	9	10	11	12
24×10	¾	99.6	116.0	23.20	19.33	16.57	14.50	12.88	11.60	10.54	9.
	1	131.3	144.4	28.88	24.06	20.63	18.05	16.04	14.44	13.12	12.
	1¼	162.1	167.6	33.52	27.93	23.94	20.95	18.62	16.76	15.23	13.
24×12	¾	109.0	150.4	30.08	25.06	21.48	18.80	16.71	15.04	13.67	12.
	1	143.8	189.6	37.92	31.60	27.08	23.70	21.06	18.96	17.23	15.
	1¼	177.7	223.0	44.60	37.16	31.85	27.87	24.77	22.30	20.27	18.
28× 8	¾	99.6	95.5	19.10	15.91	13.64	11.93	10.61	9.55	8.68	7
	1	131.3	115.0	23.00	19.16	16.43	14.37	12.77	11.50	10.45	9.
	1¼	162.1	130.5	26.10	21.75	18.64	16.31	14.50	13.05	11.86	10.
28×10	¾	109.0	130.0	26.00	21.67	18.57	16.25	14.44	13.00	11.82	10
	1	143.8	164.8	32.96	27.46	23.54	20.60	18.31	16.48	14.98	13
	1¼	177.7	188.5	37.70	31.41	26.93	23.56	20.94	18.85	17.14	15
28×12	¾	118.3	162.5	32.50	27.08	23.21	20.31	18.06	16.25	14.77	13
	1	156.3	211.8	42.36	35.30	30.26	26.48	23.53	21.18	19.25	17
	1¼	193.3	252.0	50.40	42.00	36.00	31.50	28.00	25.20	22.91	21

LINTELS OF 

SHAPES

Size, width by depth, in	Thickness of metal, in	Weight per foot, lb	C, tons	5	6	7	8	9	10	11	12
16× 6	¾	74.4	43.3	8.66	7.21	6.18	5.41	4.81	4.33	3.93	3
	1	96.9	52.4	10.48	8.73	7.48	6.55	5.82	5.24	4.76	4
	1¼	118.1	59.3	11.86	9.88	8.47	7.41	6.59	5.93	5.39	4
16× 8	¾	88.5	68.1	13.62	11.35	9.73	8.51	7.56	6.81	6.19	5
	1	115.6	83.9	16.75	13.98	11.98	10.48	9.32	8.39	7.62	6
	1¼	141.6	97.0	19.40	16.16	13.85	12.12	10.77	9.70	8.81	8
20× 8	¾	97.8	80.2	16.04	13.36	11.45	10.02	8.91	8.02	7.29	[illegible]
	1	128.1	98.7	19.74	16.45	14.10	12.33	10.96	9.87	8.97	[illegible]
	1¼	157.2	113.9	22.78	18.98	16.27	14.23	12.65	11.39	10.35	9

Table I (Continued). Safe Distributed Loads in Tons for Cast-Iron Lintels

LINTELS OF  SHAPES

Loads include weights of lintels. Maximum tensile stress 3 000 lb per sq in. See remarks, pages 622 and 623.

Size, width by depth, in	Thickness of metal, in	Weight per foot, lb	C, tons	Span in feet							
				5	6	7	8	9	10	11	12
	¾	111.9	112.0	22.40	18.66	16.00	14.00	12.44	11.20	10.18	9.33
20×10	1	146.9	139.7	27.94	23.28	19.95	17.46	15.52	13.97	12.70	11.64
	1¼	180.7	163.5	32.70	27.25	23.35	20.43	18.16	16.35	14.86	13.62
	¾	126.0	146.7	29.34	24.45	20.95	18.33	16.30	14.67	13.33	12.22
20×12	1	165.6	184.8	36.96	30.80	26.40	23.10	20.53	18.48	16.80	15.40
	1¼	204.1	218.8	43.76	36.46	31.25	27.35	24.31	21.88	19.89	18.24
	¾	107.2	91.9	18.38	15.31	13.12	11.49	10.21	9.19	8.35	7.66
24× 8	1	140.6	112.8	22.56	18.80	16.11	14.10	12.53	11.28	10.25	9.40
	1¼	172.6	130.2	26.04	21.70	18.57	16.27	14.47	13.02	11.83	10.85
	¾	121.3	127.8	25.56	21.30	18.25	15.97	14.20	12.78	11.61	10.65
24×10	1	159.4	159.5	31.90	26.58	22.78	19.94	17.72	15.95	14.50	13.29
	1¼	196.3	183.6	36.72	30.60	26.23	22.95	20.40	18.36	16.69	15.30
	¾	135.3	166.6	33.32	27.76	23.80	20.82	18.51	16.66	15.14	13.88
24×12	1	178.1	209.3	41.86	34.88	29.90	26.16	23.25	20.93	19.02	17.44
	1¼	219.7	247.7	49.54	41.28	35.39	30.96	27.52	24.77	22.51	20.64
	¾	130.7	141.4	28.28	23.57	20.20	17.67	15.71	14.14	12.85	11.78
28×10	1	171.9	177.4	35.48	29.57	25.34	22.17	19.71	17.74	16.12	14.78
	1¼	211.9	207.8	41.56	34.63	29.68	25.97	23.09	20.78	18.89	17.31
	¾	144.7	186.0	37.20	31.00	26.57	23.25	20.66	18.60	16.91	15.50
28×12	1	190.6	234.6	46.92	39.10	33.51	29.32	26.06	23.46	21.32	19.55
	1¼	235.3	277.9	55.58	46.31	39.70	34.74	30.88	27.79	25.26	23.16

3. Sections, Stresses, Buckling and Deflection of Wooden Beams and Girders

Sections and Fiber-Stresses. The cross-sections of wooden beams are almost invariably SQUARE OR RECTANGULAR, and those shapes only are considered in the following rules and formulas. Beams should have such a cross-section, that the maximum fiber-stress due to transverse bending, the maximum horizontal shear and the compression across the grain at the end-bearings do not exceed the AVERAGE ALLOWABLE UNIT STRESSES as set forth in Table XVI.

Buckling. Wooden girders should be braced laterally to prevent BUCKLING when the ratio of length to breadth exceeds twenty, or designed with a reduced fiber-stress from that allowable, where this ratio is exceeded. Tables VII to XV assume such bracing. Joists should have bridging not over 8 ft on centers. The PERCENTAGE OF REDUCTION of fiber-stress for girders should be as follows:

Ratio of length to width.....	20 to 30	30 to 40	40 to 50	50 to
Percentage of reduction......	25	34	42	50

Deflection. It is also important that beams carry the loads without DEFLECING beyond a limit fixed by the use to which the structure is applied; this lir is generally taken at 1/30 of an inch per foot of span for plastered ceilings.

3. Constants and Coefficients for Beams

Value of the Constant, A. The letter *A* in the following formulas (4) (16), denotes the SAFE LOAD for a UNIT BEAM, 1 in square in section and 1 ft span, loaded at the middle of the span. This is also one-eighteenth of t ALLOWABLE FIBER-STRESS in pounds per square inch. (See Table I, on pa 557.) The following are the values of *A*, obtained by dividing by eighteen t RECOMMENDED UNIT STRESSES for TRANSVERSE BENDING, and those given in t building laws of New York, Chicago, Baltimore and Boston.

Table II.* Coefficients for Iron, Steel and Wooden Beams. Values for A Formulas

Materials	New York	Chicago	Baltimore	Boston	Recommended
Cast iron	167	167	167	167	167
Wrought iron	667	667	667	667	667
Steel	889	889	889	889	889
Yellow pine	90	72	100	83	67
White pine	67	44	56	56	39
Spruce	67	44	75	56	39
Hemlock	44	33	....	....	33
Chestnut	....	....	....	....	44
Oak	67	67	83	56	67
Douglas fir	67	72	....	....	56

* For safe allowable working unit stresses for other woods, see Table XVI, page (From these values, *A* may be determined by dividing them by eighteen. See Ta XVII, page 648, for other stresses for woods, taken from various building laws. Tables XVIII and XIX, pages 650 and 651, for the ultimate strength of woods.

† The values of *A* for wooden beams may be increased from 30 to 40% for tempor structures, and for commercially dry and protected timber, not subject to impact, or ideal conditions.

Table III. Coefficients Recommended for Stone ‡ and Concrete Beams. Values of A

Materials	Values of *A*	Materials	Values of *A*
Granite	10	Bluestone	17
Limestone	8	Slate	22
Marble	7	Concrete 1 : 2 : 4	1.
Sandstone	6	Concrete 1 : 2 : 5	1.

‡ Values of *A* for STONE BEAMS were taken from former Building Laws of New Y and from the requirements of the Board of Fire Underwriters.

4. Flexural Strength of Wooden Beams

Section-Modulus. For beams with a rectangular cross-section, the formulas for strength can be simplified by substituting for the SECTION-MODULUS its value $\frac{1}{6} bd^2$, where b is the breadth and d the depth of the section.

Substituting this value in the general formulas for beams with rectangular cross-sections and of any material, the following formulas result:

Beams Fixed at One End and Loaded at the Other (Fig. 5).

$$\text{Safe load, in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A^*}{4 \times \text{length in feet}} \tag{4}$$

$$\text{Breadth, in inches} = \frac{4 \times \text{load} \times \text{length in feet}}{\text{square of depth} \times A^*} \tag{5}$$

Fig. 5. Cantilever Beam. Load near Free End

Fig. 6. Cantilever Beam. Distributed Load over Entire Span

Beams Fixed at One End and Loaded with a Uniformly Distributed Load (Fig. 6).

$$\text{Safe load, in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A^*}{2 \times \text{length in feet}} \tag{6}$$

$$\text{Breadth, in inches} = \frac{2 \times \text{load} \times \text{length in feet}}{\text{square of depth} \times A^*} \tag{7}$$

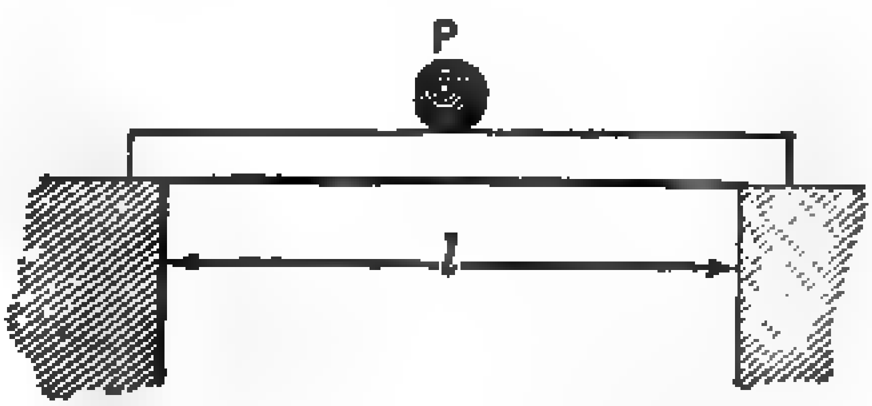


Fig. 7. Simple Beam. Load at Middle of Span

Beams Supported at Both Ends and Loaded at the Middle (Fig. 7).

$$\text{Safe load, in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A^*}{\text{span in feet}} \tag{8}$$

$$\text{Breadth, in inches} = \frac{\text{span in feet} \times \text{load}}{\text{square of depth} \times A^*} \tag{9}$$

* For values of A, see Tables II and III.

Beams Supported at Both Ends and Loaded with a Uniformly Distribu Load Over Entire Span (Fig. 8).

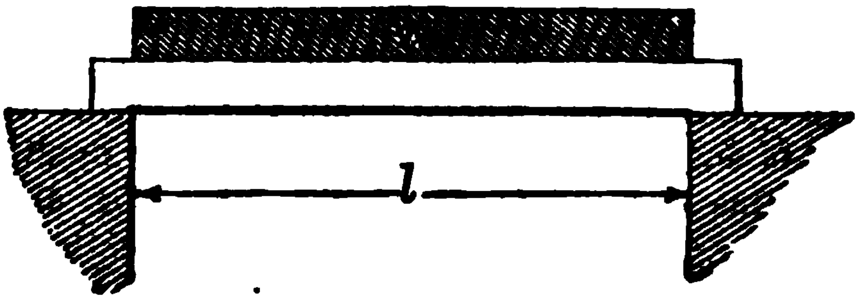


Fig. 8. Simple Beam. Distributed over Entire Span

$$\text{Safe load, in pounds} = \frac{2 \times \text{breadth} \times \text{square of depth} \times A^*}{\text{span in feet}}$$

or

$$\text{Breadth, in inches} = \frac{\text{span in feet} \times \text{load}}{2 \times \text{square of depth} \times A^*}$$

Beams Supported at Both Ends and Loaded with a Uniformly Distrib Load Over Only a Portion of the Span (Fig. 9).

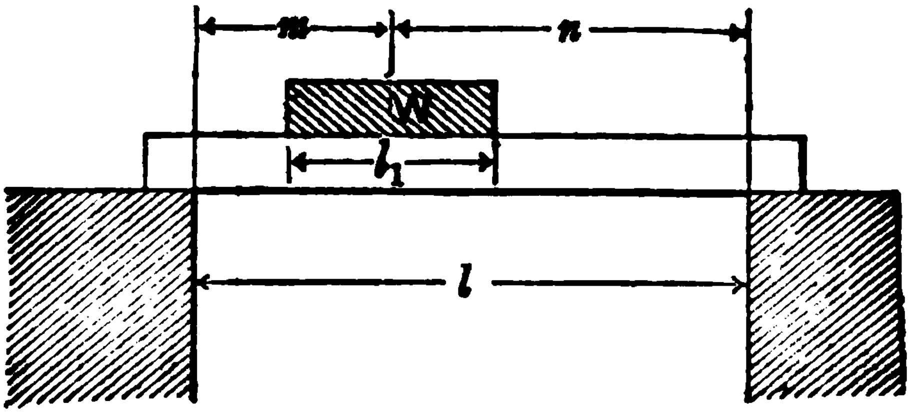


Fig. 9. Simple Beam. Distributed Load over Part of Span

In this case the dimensions of the beam required to carry the load ca accurately determined only by computing the MAXIMUM BENDING MOMEN explained in Chapter IX, and substituting the value thus found in Formula following. If, however, the length l_1 is very short in comparison with l, near the middle, then the load may be considered as CONCENTRATED at the mi of the span and the breadth of the beam may be found by Formula (9). mula (13) is used if the load is at one side of the middle. The error will b the safe side.

Beams Supported at Both Ends and Loaded with Concentrated Load, at the Middle of the Span (Fig. 10).

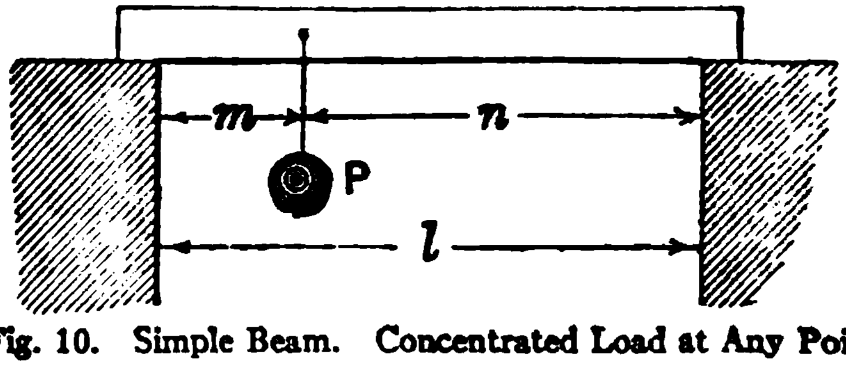


Fig. 10. Simple Beam. Concentrated Load at Any Point

$$\text{Safe load, in pounds} = \frac{\text{breadth} \times \text{square of depth} \times \text{span} \times A^*}{4 \times m \times n}$$

or

$$\text{Breadth, in inches} = \frac{4 \times \text{load} \times m \times n}{\text{square of depth} \times \text{span} \times A^*}$$

* For values of A, see Tables II and III.

Beams Supported at Both Ends and Loaded with P Pounds at a Distance m from each End (Fig. 11).

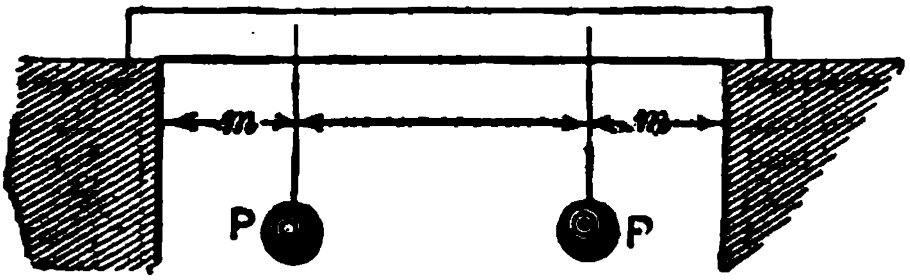


Fig. 11. Simple Beam. Two Equal Concentrated Loads Symmetrically Placed

$$\left.\begin{matrix}\text{Safe load, } P\text{, in pounds}\\ \text{at each point}\end{matrix}\right\} = \frac{\text{breadth} \times \text{square of depth} \times A^*}{4 \times m} \qquad (14)$$

$$\text{Breadth, in inches} = \frac{4 \times \text{load at one point} \times m}{\text{square of depth} \times A^*} \qquad (15)$$

Note. In the last two cases the lengths denoted by *m* and *n* should be in feet, as the spans are in feet.

8. Application of Formulas for Flexural Strength of Wooden Beams

Example 1. What load, 6 ft out from the wall, will an 8 by 14-in long-leaf yellow pine beam, securely fastened at one end into a brick wall, sustain with safety?

Solution. The safe load in pounds (Formula 4) $= \dfrac{8 \times 196 \times 67}{4 \times 6} = 4\,377$ lb

Example 2. It is desired to suspend two loads of 10 000 lb each, 4 ft from each end of an oak beam, 20 ft long. What should be the size of the beam?

Solution. Let the depth of the beam be assumed to be 16 in. Then (Formula 15)

$$\text{The breadth} = \frac{4 \times 10\,000 \times 4}{256 \times 67} = 9.3 \text{ in, nearly}$$

The beam, therefore, should be 10 by 16 in in cross-section.

Beam with Several Loads. It is required, next, to determine the size of a beam which is supported at both ends, and which will safely support several concentrated loads, or a distributed load and one or more concentrated loads. The correct method of finding the least size of a beam that will safely support a combination of loads, is to first find the MAXIMUM BENDING MOMENT, as explained in Chapter IX, page 329, and then substitute the value thus found for this BENDING MOMENT in the following formula:

$$\text{Breadth, in inches} = \frac{4 \times \text{maximum bending moment in ft-lb}}{\text{square of depth} \times A} \qquad (16)$$

A shorter and easier method is to find the EQUIVALENT DISTRIBUTED LOAD for each concentrated load, and then find the size of a beam required to support the total equivalent distributed load thus found. The equivalent distributed loads for concentrated loads applied at different proportions of the span from either end, may be obtained by multiplying the concentrated loads by the following FACTORS:

* For values of *A*, see Tables II and III.

Table IV. Factors for Equivalent Distributed Loads

	Position of load	Factor
For a concentrated load	Applied at middle of span	Multiply by 2.
For a concentrated load	Applied at one-third the span	Multiply by 1.
For a concentrated load	Applied at one-fourth the span	Multiply by 1.
For a concentrated load	Applied at one-fifth the span	Multiply by 1.
For a concentrated load	Applied at one-sixth the span	Multiply by 1
For a concentrated load	Applied at one-seventh the span	Multiply by 0
For a concentrated load	Applied at one-eighth the span	Multiply by
For a concentrated load	Applied at one-ninth the span	Multiply by 0.
For a concentrated load	Applied at one-tenth the span	Multiply by 0.

(See, also, Chapter XV, Safe Loads for Steel Beams, page 566.)

Thus, a concentrated load of 900 lb, applied at one-sixth the span from support, will result in the same maximum bending moment as a distrib load of 900 × 1⅑, or 1 000 lb.

The above method for finding the size of a beam for a combination of sev loads gives a larger beam than the correct method, by Formula (16), for reason that the maximum bending mom will not be equal to the sum of the dividual bending moments. Hence, w there are several heavy loads to be s ported, it is economical to compute maximum bending moment by the GRAI METHOD explained in Chapter IX, p 329.

Fig. 12. Girder with Three Concentrated Loads

Example 3. The girder *G*, Fig. 12, s ports the rafters of a flat roof, and three heavy beams, *A*, *B* and *C*, blocked above the roof and supporting a large t filled with water. The timber is to long-leaf yellow pine. The weight of the roof and allowance for snow is 7 lb. Each of the beams *A*, *B* and *C*, impose a load on the girder, due to weight of the tank and its contents, of 3 000 lb. What should be the size o girder?

Solution. The roof-load may be considered to be uniformly distribu The load from beam, *A*, is applied at one-third the span from one end; the from *B*, five-twelfths the span from the other end; and the load from *C*, sixth the span. The fraction five-twelfths is the mean of one-half and one-t hence the load from *B* should be multiplied by 1.89. Multiplying the centrated loads by their proper factors, the equivalent distributed load is f to be as follows:

Roof-load, distributed,	=	7 500
Load from *A*, 3 000 × 1.78	=	5 340
Load from *B*, 3 000 × 1.89	=	5 670
Load from *C*, 3 000 × 1⅑	=	3 333
Equivalent distributed load	=	21 843 lb

Assuming 16 in as the depth of the beam, and using Formula (11),

$$\text{The breadth} = \frac{12 \times 21\,843}{2 \times 256 \times 67} = 7.6 \text{ in}$$

Assuming 14 in for the depth, 10 in is obtained for the breadth; hence, the girder must be 10 by 14 in, or 8 by 16 in in cross-section.

Strut-Beams and Tie-Beams. A STRUT-BEAM is a beam that is subject to both a transverse and a compressive stress. A TIE-BEAM is one that is subject to direct tension in addition to the transverse stress. To find the strength of either, first find the size of a beam required to resist the transverse stress, and then the size of a timber, of the same depth as the beam, to resist the direct tension or compression, and add the two breadths together.

Example 4. A spruce tie-beam, 10 ft long between joints, sustains a ceiling-load of 2 000 lb and a direct tensile stress of 40 000 lb. What should be the dimensions of the beam?

Solution. As a ceiling-load is uniformly distributed, the size of the beam is determined by Formula (11), page 630. Assuming the depth to be 10 in

$$\text{The breadth} = \frac{10 \times 2\,000}{2 \times 100 \times 39}, \quad \text{or} \quad 2\tfrac{1}{2} \text{ in, nearly}$$

The resistance of spruce to tension (see Table XVI, page 647) is 800 lb per sq in. 40 000/800 = 50 sq in, which is equivalent to a 5 by 10-in section. It will require, therefore, a beam 7½ by 10 in in cross-section to resist both the transverse stress and the direct tension. If the tie-beam is cut in any way so as to reduce the section, except over a support, the dimensions must be increased accordingly.

Example 5. A strut-beam of white pine, 10 ft long, supports a distributed roof-load of 6 000 lb, and is also subject to a direct compression of 64 000 lb. What should be the size of the beam?

Solution. Assuming 14 in for the depth, the breadth for the transverse load is found by Formula (11), page 630

$$\text{The breadth} = \frac{10 \times 6\,000}{2 \times 196 \times 39} = 3.9 \text{ in, nearly}$$

Using Formula (4), page 450, from which is computed Table IV, page 452, giving the safe loads for white-pine posts, it is found that a 7½ by 14-in post, 10 ft long will safely carry the compressive stress, 64 000 lb. Hence it will require a 7½ by 14-in beam to resist the compressive stress, and a 4 by 14-in beam to resist the transverse load. The beam, therefore, should be 12 by 14 in in cross-section to resist them both.

6. Relative Strengths of Beams

Relative Strengths of Rectangular Beams. From an inspection of the foregoing formulas it is found that the RELATIVE STRENGTHS of beams of rectangular cross-sections, for the different cases is as shown in Table V.

Strengths of Beams of Any Constant Cross-Section. The STRENGTH-RATIOS given in Table V are true for beams of any constant cross-section of whatever form.

Beam on Edge. When a beam of square cross-section is supported on its edge, that is, when one of its diagonals is vertical, it will bear about seven-tenths as great a breaking-load as it will when it is supported on one side.

Table V. Relative Strengths of Rectangular Beams

Kind of load	Position of load	Strength ratios
Beam supported at both ends		
Uniformly distributed	Over entire span	1
Concentrated	At middle of span	1/2
Concentrated	At one-third the span	9/16
Concentrated	At one-fourth the span	2/3
Concentrated	At one-fifth the span	25/32
Concentrated	At one-sixth the span	9/10
Concentrated	At one-seventh the span	49/48
Concentrated	At one-eighth the span	8/7
Concentrated	At one-ninth the span	81/64
Concentrated	At one-tenth the span	25/18
Beam fixed at one end, or cantilever beams		
Uniformly distributed	Over entire span	1/4
Concentrated	At the free end	1/8
Beam supported at one end and fixed at the other end		
Uniformly distributed	Over entire span	1
Concentrated	Near the middle of span	13/20
Beam fixed at both ends		
Uniformly distributed	Over entire span	1 1/2
Concentrated	At middle of span	1

The Strongest Beam Cut From a Cylindrical Log is one in which breadth is to the depth as 5 is to 7, very nearly, and the dimensions of such a b can be found graphically, as shown in Fig. 13. Any diagonal, as *ab*, is di and divided into three equal parts by the points *c* *d*; from these points lines perpendicular to *ab* are dr and the points *e* and *f* connected with *a* and *l* shown.

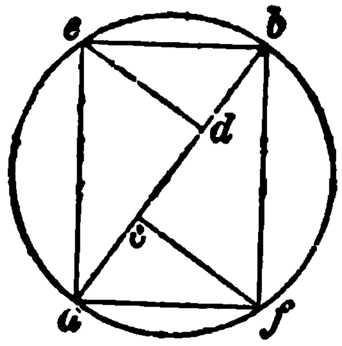


Fig. 13. Strongest Beam of Rectangular Section Cut from Log

Cylindrical Beams. A CYLINDRICAL BEAM is ten-seventeenths as strong as a beam with a sq cross-section, the side of the square being equal to diameter of the circular section of the cylindrical b Hence, to find the safe load for a cylindrical beam, find the proper load for the corresponding sq section beam, and divide this load by 1.7.

The Bearing of the Ends of a Beam on a beyond a certain distance does not strengthen the beam. In general, a l should have a bearing of 4 in, or if it is very long, 6 in.

The Weight of the Beam Itself. The formulas given for the strengt beams do not take into account the WEIGHT OF THE BEAMS THEMSELVES, hence the safe loads of the formulas include both the external loads and weights of the material in the beams. In small wooden beams, the weig

each beam is generally so small, compared with the external load, that it need not be taken into account. But for larger wooden beams, and for metal and stone beams, the weight of the beam should be subtracted from the safe load if the load is distributed; and if the load is applied at the middle, one-half the weight of the beam should be subtracted.

The Weight of Timber. The weight per cubic foot for different kinds of timber may be found in the table in Part III, pages 1501 to 1508, giving the Weights of Various Substances.

7. Tables for Strength and Stiffness of Wooden Beams

Tables VII to XV for the Strength and Stiffness of Wooden Beams are given on pages 638 to 646, for BEAMS ONE INCH IN BREADTH. To find the length for any other breadth, multiply the proper tabular value by the breadth of the beam in inches. To obtain the required breadth for any load, divide the given load in pounds by the proper tabular value. In heading the tables, prominence has been given to the values used for S, and the corresponding values of A, so that those who prefer to use for any wood a value different from that recommended, need only to look up the table based on the value they desire to employ. For certain cases and in some cities, the building laws specify 1 300, 1 500 and 1 800 pounds as values of S to be used for long-leaf yellow pine; hence Tables XIII, XIV and XV, based on these values, are added.

Since timber is weak in HORIZONTAL SHEAR compared with its strength in TENSION and COMPRESSION, the safe load a beam of short span can carry is governed, not by its resistance to CROSS-BREAKING, but by its RESISTANCE to SHEARING along the NEUTRAL SURFACE. Wooden beams and joists, therefore, should be dimensioned to safely withstand this SHEARING ACTION. The ratio of the SHEARING to the FLEXURAL STRENGTH is not exactly the same for different kinds of wood, but for practical use and in the tables it has been assumed to be one-twelfth of the WORKING UNIT FIBER-STRESS. As it can be shown * that the ratio of the span to the depth of a rectangular beam, uniformly loaded, is directly proportional to its CROSS-BREAKING STRESS and SHEARING WORKING STRESS, the tabular loads are figured for the PERMISSIBLE UNIT FIBER-STRESS, where the length of the span is twelve or more times the depth of the beam; while for shorter lengths, the tabular loads are governed by the SHEAR. To determine the safe load on beams for a deflection not exceeding 1/360 of the span, tabular values have been placed directly underneath the safe loads for strength. These values are based on the MODULUS OF ELASTICITY, E, given in the tables.

The FORMULA FOR FLEXURE used in determining the safe uniformly distributed loads in the tables is (see Formulas (1), page 333 and (2)′, page 557)

$$M = \frac{SI}{c} = \frac{Sbd^2}{6} = \frac{Wl}{8}$$

hence

$$W = \frac{4}{3}\frac{bd^2S}{l}, \text{ in which } l \text{ is the span in inches}$$

The FORMULA FOR SHEAR is

$$W = \frac{4\,bdS_s}{3}$$

* Materials of Construction, J. B. Johnson, page 55.

The FORMULA FOR DEFLECTION is (see, also, Formulas (1) to (17) and Tal
Chapter XVIII)

$W = \dfrac{Ed^3}{8\,100\,l^2}$ in which l is the span in feet;
M = maximum bending moment in inch-pounds;
I = moment of inertia of the cross-section of the beam in biquadr
inches;
$c = d/2$ = one-half the depth of the beam in inches;
SI/c = resisting moment of the cross-section in inch-pounds;
W = total safe load in pounds, uniformly distributed;
b = breadth of the beam in inches;
d = depth of the beam in inches;
l = span, in feet or inches, as noted for the different formulas;
S = unit flexural fiber-stress in pounds per square inch;
$S_s = S/12$ = horizontal unit shearing-stress, in pounds per square inch, along
neutral surface;
E = modulus of elasticity in pounds per square inch.

Example 6. What is the safe, uniformly distributed load, correspondin
a fiber-stress of 1 500 lb per sq in, for an 8 by 14-in long-leaf yellow-pine b
supported at both ends, and having a 24-ft clear span?

Solution. From Table XIV, the load for a 1-in thickness is 1362 lb. He
1 362 × 8 = 10 896 lb, the total load for the beam. If the deflection of
beam should not be more than 1/360 of the span, the safe load for 1-in thick
should not exceed 882 lb. Hence, 882 × 8 = 7 056 lb, is the maximum loa
be used in this case. It is assumed that 1 500 lb per sq in is allowed for

Example 7. What should be the size of a Norway-pine beam required to c
a distributed load of 6 400 lb over a clear span of 18 ft?

Solution. From Table X, it is found that a beam 12 in deep and 1 in
and with an 18-ft span, will support 711 lb. Dividing the load, 6 400 lb
711, the result is 9 for the breadth of the beam in inches. Hence the b
should be 9 by 12 in, to carry a distributed load of 6 400 lb over a span of
As the deflection-load of 593 lb can be increased 20% for Norway pine, the b
is safe for deflection; if, however, cypress is used, 593 must be taken in pla
711, to determine the breadth of the beam. This would result in a bea
by 12 in.

Different Positions of Loads. To find the safe load, concentrated a
middle of the span of a given beam, find the safe distributed load, as in Exa
6, and divide this load by 2. To find the safe load concentrated at
point other than the middle of the span, find the safe distributed load fo
given span, and divide this load by the proper factor taken from Tabl
page 632. To find the size of a beam to support a given concentrated
multiply the given load by the factor corresponding to the position of the
as given in Table IV, and then proceed as in Example 7.

Use of Formulas. If in doubt as to the application of the tables, in s
cases, use one of the formulas, from (4) to (16), applying to the case.
formulas and tables should always give the same result.

Nominal and Actual Sizes of Beams. The tables may be used for
the dimensions of which are less than the NOMINAL DIMENSIONS. D
beams, and, in many localities, floor-joists carried in stock, are more
scant of the nominal dimensions, and for such beams and joists a reduct
the safe load must be made to correspond with the reduction in size.

...SED SIZES are generally 1/4 in scant, up to 4 in in breadth, above which they ...e 1/2 in scant; while in depth they are all generally 1/2 in less than the nominal ...e. The safe loads may be obtained by multiplying the safe loads for the ...responding nominal sizes, as given in Tables VII to XV, by the factors given ...the following table.

Table VI. Conversion Factors for Actual Sizes of Wooden Beams

Cross-sections of beams in inches	Factors	Cross-sections of beams in inches	Factors
1¾×5½	1.47	1¾×11½	1.61
2¾×5½	2.31	2¾×11½	2.53
1¾×6½	1.51	1¾×13½	1.63
2¾×6½	2.51	2¾×13½	2.56
1¾×7½	1.54	1¾×15½	1.65
2¾×7½	2.42	2¾×15½	2.58
1¾×9½	1.53	1¾×17½	1.65
2¾×9½	2.48	2¾×17½	2.60

...ample 8. What is the safe load for a 2¾ by 13½-in spruce beam, with ...18-ft span?

...lution. From Table VIII, the safe load for a 1 by 14-in beam is 847 lb. ...ltiplying this by 2.56, we have 2 178 lb as the safe distributed load for a ...m 2¾ by 13½ in in cross-section. For a full 3 by 14-in cross-section, the ...e load would be 2 541 lb.

Stone Beams. The above formulas may be used for rectangular stone ...ms when the proper coefficients, recommended in Table III, page 628, are ...ted. Sandstone beams should never be subjected to any heavy loads and ...dstone lintels should be relieved by steel beams or by brick arches over them ...back of them.

Concrete Beams are generally reinforced with steel rods, but when used ...out reinforcement, the coefficient, A, given in Table III, is recommended.

...e of Tables VII to XV. The safe loads given in Tables VII to XV ...correct for the fiber-stresses indicated; but for greater convenience in using ...tables, each figure in the units-place of each value may be made a cipher, ...each figure in the tens-place may be increased by one when the unit-figure ...5 or greater. Thus, 505 would be 500, 506 would be 510, etc.

...portant Notes on Stresses and Loads for Wooden Beams. In compiling and ...the tables of safe loads for wooden beams, the following important considerations ...be kept in mind:

...) Unseasoned timber is very much weaker than commercially dry timber, that is, ...er containing from 10 to 15% of moisture.
...) Timber containing large or loose knots is much weakened.
...) When impact has to be considered, the stresses should be reduced.
...) For continuous, heavy loading, relatively low stresses should be used.
...) Commercial dimensions are smaller than nominal dimensions.
...) Timbers deteriorate and the factors of safety for strength grow smaller with time.
...) The modulus of elasticity, E, for unseasoned timber, should be reduced 50% from ...lue given for thoroughly seasoned timber.
...) It is better engineering practice to compute tables of safe loads based on conserva-...stresses for average or poor conditions, increasing the values given when conditions ...ideal, than to recommend values for ideal conditions which usually do not exist. ...notes, pages 628 and 647, regarding increase in the table-values.) Editor-in-Chief.

Safe Distributed Loads * in Pounds for Rectangular Wooden Bea
verage Hemlock. Maximum Fiber-Stress, $S = 600$ lb per sq in.
$E = 900\,000$ lb per sq in. $A = 33$

The first horizontal line gives the depth of the beam in inches
The loads are for beams one inch wide and supported at both ends

6	8	10	12	14	16	18
400	533	666	800	933	1 066	1 200
343	533	666	800	933	1 066	1 200
300	533	666	800	933	1 066	1 200
266	474	666	800	933	1 066	1 200
240 240	427	666	800	933	1 066	1 200
218 199	388	605	800	933	1 066	1 200
200 166	356	555	800	933	1 066	1 200
185 143	328	513	738	933	1 066	1 200
171 122	305 291	477	686	933	1 066	1 200
160 107	285 253	445	640	871	1 066	1 200
150 94	267 222	417	600	817	1 066	1 200
....	251 197	392 384	565	762	1 003	1 20[illegible]
....	237 175	371 343	534	726	948	1 20[illegible]
....	225 157	351 308	505	688	898	1 13[illegible]
....	213 142	333 277	480 480	653	854	1 08[illegible]
....	....	317 252	462 435	623	813	1 02[illegible]
....	....	303 229	436 397	594	776	98[illegible]
....	....	290 211	417 363	568	742	93[illegible]
....	....	278 193	400 334	545 529	712	90[illegible]
....	....	....	384 308	523 488	683	86[illegible]
....	....	....	369 284	503 452	657	83[illegible]
....	....	....	356 264	482 418	633 625	8[illegible]
....	....	....	343 245	467 389	609 582	7[illegible]
....	....	....	....	451 353	589 542	7[illegible]
.....	....	....	....	436 339	569 505	[illegible]

ove zigzag lines calculated for horizontal shear. Where two loads are
calculated for strength, the lower for deflection not to exceed 1/360 the
to 40% to strength-values for ideal conditions. See notes, pages 6?8.

Table VIII. Safe Distributed Loads * in Pounds for Rectangular Wooden Beams For Average White Pine, Spruce and Eastern Fir. Maximum Fiber-Stress, $S = 700$ lb per sq in. E † = 1 000 000 lb per sq in. $A = 39$

The first horizontal line gives the depth of the beam in inches
The loads are for beams one inch wide and supported at both ends

Span in feet	6	8	10	12	14	16	18
6	467	622	777	933	1 089	1 244	1 400
7	400	622	777	933	1 089	1 244	1 400
8	350	622	777	933	1 089	1 244	1 400
9	311	552	777	933	1 089	1 244	1 400
10	280 267	497	777	933	1 089	1 244	1 400
11	255 221	453	707	933	1 089	1 244	1 400
12	233 185	415	648	933	1 089	1 244	1 400
13	216 158	383 374	598	861	1 089	1 244	1 400
14	200 136	356 323	556	800	1 089	1 244	1 400
15	187 119	332 281	518	747	1 016	1 244	1 400
16	175 104	311 247	486 482	700	952	1 244	1 400
17	....	293 219	458 427	660	897	1 172	1 400
18	....	276 195	433 381	623	847	1 107	1 400
19	....	262 175	410 342	590	802	1 048	1 326
20	....	....	389 308	560 534	762	996	1 260
21	....	....	370 280	534 484	726	948	1 200
22	....	....	354 255	509 441	692	906	1 144
23	....	....	338 234	487 403	662 641	866	1 096
24	....	....	324 215	468 371	635 588	830	1 050
25	....	....	....	448 342	610 542	796	1 008
26	....	....	....	430 316	586 502	766 750	970
27	....	....	....	415 293	565 465	738 695	934
28	....	....	....	400 272	544 432	711 646	900
29	....	....	....	....	526 403	687 602	868 856
30	....	....	....	....	508 377	664 562	840 800

* Add 30 to 40% to strength-values for ideal conditions. See notes, pages 628, 637.

† For first-class, dry spruce and Eastern fir, $E = 1\ 200\ 000$ could safely be used, making safe deflection-loads those given in Table XI. See, also, foot-note with Table VII

**Table IX. Safe Distributed Loads * in Pounds for Rectangular Wooden Bea[ms]
For Average California Red Wood and Cedar. Maximum Fiber-Stress,
$S = 750$ lb per sq in. $E = 700\,000$ lb per sq in. $A = 41.7$**

Span in feet	6	8	10	12	14	16	18
	The first horizontal line gives the depth of the beam in inches. The loads are for beams one inch wide and supported at both ends						
6	500	667	833	1 000	1 167	1 333	1 500
7	423 382	667	833	1 000	1 167	1 333	1 500
8	375 292	667	833	1 000	1 167	1 333	1 500
9	333 231	592 547	833	1 000	1 167	1 333	1 500
10	300 187	533 443	833	1 000	1 167	1 333	1 500
11	274 155	485 366	757 714	1 000	1 167	1 333	1 500
12	250 130	445 307	641 600	1 000	1 167	1 333	1 500
13	231 110	410 262	641 512	923 885	1 167	1 333	1 500
14	214 95	382 226	595 441	857 763	1 167	1 333	1 500
15	....	356 197	556 384	800 665	1 088 1 060	1 333	1 500
16	....	333 173	521 337	750 584	1 020 929	1 333	1 500
17	....	....	491 299	706 518	961 822	1 254 1 223	1 500
18	....	....	463 267	667 462	908 733	1 184 1 090	1 500
19	....	....	439 240	632 414	860 658	1 122 982	1 421 1 396
20	....	....	....	600 374	816 594	1 066 886	1 350 [illegible]
21	....	....	....	572 339	778 526	1 016 803	[illegible] [illegible]
22	....	....	....	547 309	742 491	970 732	1 227 [illegible]
23	....	....	....	522 282	710 448	928 670	1 174 954
24	....	....	....	500 260	681 412	890 616	[illegible] [illegible]
25	....	....	....	480 239	653 380	854 567	[illegible] [illegible]
26	....	....	....	463 221	628 351	821 525	[illegible] [illegible]
27	....	....	....	444 205	605 326	791 487	[illegible] [illegible]
28	....	....	....	428 190	583 303	762 452	[illegible] [illegible]
29	....	....	....	....	563 282	736 421	[illegible] [illegible]
30	....	....	....	....	544 264	712 393	[illegible] [illegible]

* Add 30 to 40% to strength-values for ideal conditions. See notes, pages 6[illegible] 647. See, also, foot-note with Table VII.

Table X. Safe Distributed Loads * in Pounds for Rectangular Wooden Beams
For Average Norway Pine, Cypress and Chestnut.
Maximum Fiber-Stress, S = 800 lb per sq in. E † = 900 000 lb per sq in. A = 4

Span in feet	The first horizontal line gives the depth of the beam in inches. The loads are for beams one inch wide and supported at both ends						
	6	8	10	12	14	16	18
6	533	711	889	1 066	1 244	1 422	1 600
7	457	711	889	1 066	1 244	1 422	1 600
8	400 375	711	889	1 066	1 244	1 422	1 600
9	356 296	632	889	1 066	1 244	1 422	1 600
10	320 240	569 569	889	1 066	1 244	1 422	1 600
11	291 199	517 470	809	1 066	1 244	1 422	1 600
12	267 166	474 395	742	1 066	1 244	1 422	1 600
13	246 142	438 337	684 658	985	1 244	1 422	1 600
14	229 122	407 291	635 567	914	1 244	1 422	1 600
15	214 107	379 253	593 494	854 854	1 161	1 422	1 600
16	200 94	356 222	556 432	800 750	1 089	1 422	1 600
17	....	335 197	524 384	754 665	1 025	1 339	1 600
18	....	316 175	494 343	711 593	968 914	1 264	1 600
19	....	300 157	468 308	674 532	917 846	1 198	1 517
20	....	284 142	445 277	640 480	871 752	1 138 1 138	1 441
21	....	....	423 252	609 435	830 692	1 084 1 032	1 372
22	....	....	404 229	582 397	792 630	1 035 941	1 309
23	....	....	387 211	557 363	758 577	990 860	1 253 1 225
24	....	....	371 193	534 334	726 529	949 790	1 200 1 126
25	....	....	....	512 308	697 488	911 728	1 152 1 037
26	....	....	....	492 284	670 452	876 675	1 108 960
27	....	....	....	474 264	646 418	843 625	1 068 890
28	....	....	....	457 245	622 389	813 582	1 029 827
29	....	....	....	....	601 353	785 542	993 770
30	....	....	....	....	581 339	759 506	960 720

* Add 30 to 40% to strength-values for ideal conditions. See notes, pages 628, 637, 647 also foot-note with Table VII.
† For safe deflection-loads for Norway pine, add 20% to the above values.

fe Distributed Loads * in Pounds for Rectangular Wooden Bea)ouglas Fir and Short-Leaf Yellow Pine. Fiber-Stress, S = 1 000 l per sq in. E † = 1 200 000 lb per sq in. A = 55.6

The first horizontal line gives the depth of the beam in inches
The loads are for beams one inch wide and supported at both ends

6	8	10	12	14	16	18
567	889	1 111	1 333	1 556	1 778	2 000
571	889	1 111	1 333	1 556	1 778	2 000
500 500	889	1 111	1 333	1 556	1 778	2 000
444 395	790	1 111	1 333	1 556	1 778	2 000
400 320	711	1 111	1 333	1 556	1 778	2 000
364 265	647 628	1 010	1 333	1 556	1 778	2 000
333 222	593 527	926	1 333	1 556	1 778	2 000
308 190	547 449	855	1 231	1 556	1 778	2 000
286 163	508 388	794 757	1 143	1 556	1 778	2 000
267 143	474 337	741 659	1 067	1 452	1 778	2 000
250 125	445 296	695 578	1 000 1 000	1 361	1 778	2 000
......	419 263	654 512	942 886	1 281	1 674	2 000
......	395 234	618 457	890 790	1 210	1 581	2 000
......	374 210	585 410	843 710	1 146 1 126	1 498	1 895
......	356 190	556 370	800 641	1 088 1 016	1 423	1 800
......	....	528 336	762 581	1 037 922	1 355	1 71
......	....	505 306	727 529	990 841	1 293 1 254	1 63
......	....	483 281	696 484	947 770	1 237 1 147	1 56
......	....	463 258	667 445	908 706	1 186 1 053	1 50 1 50
......	....	....	640 410	871 650	1 138 972	1 44 1 38
......	....	....	615 380	838 602	1 094 900	1 38 1 28
......	....	....	593 352	807 558	1 054 834	1 33 1 18
......	....	....	572 327	778 518	1 016 776	1 28 1 1
......	....	....	....	751 484	982 725	1 24 1 0
......	....	....	....	726 452	949 674	1 2 9

40% to strength-values for ideal conditions. See notes, pages 628, 63
:tion-loads for Douglas fir, add 25%. See, also, foot-note with Tabl

Table XII. Safe Distributed Loads * in Pounds for Rectangular Woo
For Average White Oak and Long-Leaf Yellow Pine†. Maximum
Stress, S = 1200 lb per sq in. E = 1 500 000 lb per sq in. A =

Span in feet	The first horizontal line gives the depth of the beam in i: The loads are for beams one inch wide and supported at bo					
	6	8	10	12	14	16
6	800	1 067	1 333	1 600	1 867	2 133
7	686	1 067	1 333	1 600	1 867	2 133
8	600	1 067	1 333	1 600	1 867	2 133
9	533 495	949	1 333	1 600	1 867	2 133
10	480 400	854	1 333	1 600	1 867	2 133
11	437 332	776	1 212	1 600	1 867	2 133
12	400 278	711 658	1 111	1 600	1 867	2 133
13	369 247	656 561	1 026	1 477	1 867	2 133
14	343 204	610 485	953 946	1 371	1 867	2 133
15	320 179	569 422	890 824	1 280	1 741	2 133
16	300 156	533 371	834 724	1 200	1 633	2 133
17	....	502 329	785 642	1 130 1 108	1 537	2 009
18	....	474 293	741 572	1 067 990	1 452	1 898
19	....	449 263	702 513	1 010 886	1 375	1 795
20	....	426 237	666 462	960 802	1 306 1 272	1 708
21	....	....	634 420	914 726	1 245 1 154	1 626
22	....	....	606 383	872 662	1 188 1 051	1 552
23	....	....	579 351	835 605	1 136 962	1 484 1 435
24	....	....	556 322	800 557	1 090 882	1 423 1 318
25	....	....	....	768 513	1 045 813	1 366 1 215
26	....	....	....	738 473	1 006 753	1 313 1 125
27	....	....	....	711 440	969 698	1 265 1 043
28	....	....	....	686 410	933 648	1 218 970
29	....	....	....	....	902 605	1 178 903
30	....	....	....	....	871 566	1 138 843

* Add 30 to 40% to strength-values for ideal conditions. See notes, pag 647. See, also, foot-note with Table VII.

† For safe loads for fiber-stresses of 1300, 1500 and 1800 lb per sq in, see T XIV and XV, respectively.

Table XIII. Safe Distributed Loads in Pounds for Rectangular Wooden Bea

Maximum Fiber-Stress, S = 1 300 lb per sq in. A = 72.2

Span in feet	6	8	10	12	14	16	18
6	867	1 155	1 444	1 733	2 022	2 311	2 600
7	743	1 155	1 444	1 733	2 022	2 311	2 600
8	650	1 155	1 444	1 733	2 022	2 311	2 600
9	567	1 027	1 444	1 733	2 022	2 311	2 600
10	520	924	1 444	1 733	2 022	2 311	2 600
11	473	840	1 311	1 733	2 022	2 311	2 600
12	433	770	1 200	1 733	2 022	2 311	2 600
13	400	711	1 111	1 600	2 022	2 311	2 600
14	371	660	1 032	1 486	2 022	2 311	2 600
15	347	616	963	1 387	1 887	2 311	2 600
16	325	578	903	1 300	1 770	2 311	2 600
17	....	544	849	1 224	1 664	2 175	2 600
18	....	514	802	1 156	1 572	2 054	2 600
19	....	487	760	1 095	1 490	1 946	2 463
20	....	462	722	1 040	1 415	1 849	2 340
21	....	....	688	990	1 348	1 761	2 229
22	....	....	657	945	1 286	1 681	2 127
23	....	....	628	904	1 230	1 608	2 035
24	....	....	602	867	1 179	1 541	1 950
25	....	....	....	832	1 132	1 479	1 872
26	....	....	....	800	1 088	1 422	1 800
27	....	....	....	770	1 048	1 369	1 733
28	....	....	....	743	1 011	1 321	1 671
29	....	....	....	....	976	1 275	1 614
30	....	....	....	....	943	1 232	1 560

The first horizontal line gives the depth of the beam in inches. The loads are for beams one inch wide and supported at both ends.

Loads above the heavy, black zigzag lines are calculated for resistance to shear.

For safe deflection-loads, see values in Tables VII to XII, according to the value of used, and determined by the deflection-formula, page 636.

Table XIV. Safe Distributed Loads in Pounds for Rectangular Wooden Beams

Maximum Fiber-Stress, S = 1 500 lb per sq in. A = 83.3

Span in feet	The first horizontal line gives the depth of the beam in inches. The loads are for beams one inch wide and supported at both ends						
	6	8	10	12	14	16	18
6	1 000	1 333	1 667	2 000	2 333	2 667	3 000
7	857	1 333	1 667	2 000	2 333	2 667	3 000
8	750	1 333	1 667	2 000	2 333	2 667	3 000
9	667	1 185	1 667	2 000	2 333	2 667	3 000
10	600	1 067	1 667	2 000	2 333	2 667	3 000
11	548	970	1 515	2 000	2 333	2 667	3 000
12	500	890	1 390	2 000	2 333	2 667	3 000
13	462	820	1 282	1 846	2 333	2 667	3 000
14	428	764	1 190	1 714	2 333	2 667	3 000
15	....	712	1 112	1 600	2 178	2 667	3 000
16	....	667	1 042	1 500	2 042	2 667	3 000
17	....	....	982	1 412	1 974	2 510	3 000
18	....	....	926	1 334	1 815	2 370	3 000
19	....	....	878	1 264	1 720	2 246	2 842
20	....	....	....	1 200	1 632	2 133	2 700
21	....	....	....	1 144	1 556	2 032	2 571
22	....	....	....	1 094	1 484	1 940	2 455
23	....	....	....	1 044	1 420	1 856	2 348
24	....	....	....	1 000	1 362	1 780	2 250
25	....	....	....	960	1 306	1 708	2 [illegible]
26	....	....	....	926	1 256	1 642	2 076
27	....	....	....	888	1 210	1 582	2 000
28	....	....	....	856	1 166	1 524	1 930
29	....	....	....	....	1 126	1 472	1 862
30	....	....	....	....	1 088	1 422	1 800

Loads above the heavy, black zigzag lines are calculated for resistance to shear.

For safe deflection-loads, see values in Tables VII to XII, according to the value of E used, and determined by the deflection-formula, page 636.

Table XV. Safe Distributed Loads in Pounds for Rectangular Wooden Beam

Maximum Fiber-Stress, $S = 1\,800$ lb per sq in. $A = 100$

Span in feet	The first horizontal line gives the depth of the beam in inches. The loads are for beams one inch wide and supported at both ends						
	6	8	10	12	14	16	18
6	1 200	1 600	2 000	2 400	2 800	3 200	3 600
7	1 030	1 600	2 000	2 400	2 800	3 200	3 600
8	900	1 600	2 000	2 400	2 800	3 200	3 600
9	800	1 422	2 000	2 400	2 800	3 200	3 600
10	720	1 280	2 000	2 400	2 800	3 200	3 600
11	655	1 164	1 818	2 400	2 800	3 200	3 600
12	600	1 067	1 667	2 400	2 800	3 200	3 600
13	554	985	1 539	2 215	2 800	3 200	3 600
14	514	914	1 428	2 057	2 800	3 200	3 600
15	480	853	1 333	1 920	2 613	3 200	3 600
16	450	800	1 250	1 800	2 450	3 200	3 600
17	....	753	1 176	1 694	2 306	3 012	3 600
18	....	711	1 111	1 600	2 178	2 844	3 600
19	....	674	1 053	1 516	2 063	2 695	3 411
20	....	640	1 000	1 440	1 960	2 560	3 240
21	....	....	....	1 371	1 867	2 438	3 086
22	....	....	....	1 309	1 782	2 327	2 945
23	....	....	....	1 252	1 704	2 226	2 817
24	....	....	....	1 200	1 633	2 133	2 700
25	....	....	....	1 152	1 568	2 048	2 592
26	....	....	....	1 108	1 508	1 969	2 492
27	....	....	....	1 067	1 452	1 896	2 400
28	....	....	....	1 029	1 400	1 829	2 314
29	....	....	....	....	1 352	1 766	2 235
30	....	....	....	....	1 307	1 707	2 160

Loads above the heavy, black zigzag lines are calculated for resistance to shear.

For safe deflection-loads, see values in Tables VII to XII, according to the value of used, and determined by the deflection-formula, page 636.

8. Working Unit Stresses for Average, Unseasoned Woods

Safe Working Unit Stresses for unseasoned woods (except for E) are given in Table XVI. They are compiled and adapted largely from recommended UNIT STRESSES adopted by the Association of Railway Superintendents of Bridges and Buildings and by the American Railway Engineering Association. (See, also, page 449.)

Table XVI. Safe Working * Unit Stresses for Unseasoned Woods, in Pounds per Square Inch

Kind of wood	Tension		Compression			Bending †		Shearing	
	With the grain§	Across the grain	With the grain: End-bearing	With the grain: Columns‡ under 15 diams	Across the grain	Extreme fiber-stress§	Modulus of elasticity,‖ $E/1\,000$	With the grain	Across the grain
Factor of safety	Ten	Ten	Five	Five	Four	Six	One	Four	Four
White oak	1 200	200	1 400	1 000	500	1 200	1 500	200	1 000
White pine	700	50	1 100	800	200	700¶	1 000	100	500
Long-leaf yellow pine	1 200	60	1 400	1 000	350	1 200	1 500	150	1 250
Douglas fir	800	....	1 200	900	200	800¶	1 500	130	900
Short-leaf yellow pine	900	50	1 100	800	250	1 000	1 200	100	1 000
Red pine and Norway pine	800	50	1 000	750	200	800	1 100	....	750
Spruce and eastern fir	800	50	12 00	900	200	700¶	1 200	100	750
Hemlock	600	....	1 100	800	150	600	900	100	600
Cypress	600	....	1 000	750	200	800	900	....	....
Cedar	700	....	1 100	750	200	700	700	100	400
Chestnut	850	....	....	800	250	800	1 000	150	500
Cal. red wood	700	....	900	800	150	750	700	100	....
Cal. spruce	....	....	....	800	....	800	1 200	....	....

* The stresses given, except for E, may be increased 30% for protected, commercially dry timber, not subject to impact, as in most buildings.

† See also, Table I, page 557, Table XVII, page 648, and Table I, page 1138.

‡ The larger end-bearing stresses are frequently used for short columns and for column-formulas. (See tables, pages 449, 1138.) Lower factors of safety give higher stresses.

§ Some of these values are considered too low, relatively, by some building codes.

‖ These values of E are for seasoned timber. For unseasoned timber, reduce E [illegible]%.

¶ The New York Building Code (1917) stresses for these are 1 200 lb per sq in.

9. Working Unit Stresses for Woods. Taking from Building Laws.

The Allowable Working Unit Stresses for different woods, taken from the building laws of four cities, are given in Table XVII. The UNIT STRESSES are for TENSION, COMPRESSION, BENDING and SHEAR.

Table XVII. Working Unit Stresses for Woods, in Pounds per Square Inc

Kind of stress	Kind of wood	New York *	Chicago	Baltimore §	Bosto
Tension	Yellow pine†	1 200	1 300	1 800LLYP	....
	White pine	700	800	1 000	....
	Spruce‡	800	800	1 200	....
	Hemlock	600	800	800	....
	Douglas fir	800	1 300	....	....
	Oak	1 200	1 200	1 500	....
	Locust	......	....	....	....
	....	......	1 000SLYP	1 200VP	..
Compression with the grain	Yellow pine†	1 600	1 100	1 000LLYP	1 60
	White pine	1 000	700	800	1 00
	Spruce‡	1 200	700	800	1 00
	Hemlock	800	500	600	....
	Douglas fir	1 200	1 100	....	1 50
	Oak	1 400	900	1 000	1 40
	Locust	1 200	....	1 200	....
	....	. .	800SLYP‖	800NC or YP	....
Compression across the grain	Yellow pine†	350	250	600LLYP	50
	White pine	250	200	400	25
	Spruce‡	200	200	400	25
	Hemlock	150	150	500	....
	Douglas fir	200	....	....	40
	Oak	500	500	600	60
	Locust	......	....	1 000	....
	....	... ..	250SLYP‖	400NC or VP	....
Transverse bending	Yellow pine†	1 600	1 300	1 800LLYP	1 60
	White pine	1 200	800	1 000	1 00
	Spruce‡	1 200	800	1 350	1 00
	Hemlock	800	600	1 000	....
	Douglas fir	1 200	1 300	....	1 50
	Oak	1 200	1 200	1 500	1 40
	Locust	......	....	....	....
	....	......	1 000SLYP‖	....	....
Shear with the grain	Yellow pine†	150	130	100LLYP	15
	White pine	100	80	85	10
	Spruce‡	100	80	90	10
	Hemlock	100	60	75	....
	Douglas fir	100	130	....	12
	Oak	200	200	100	15
	Locust	......	....	....	...
	....	......	120SLYP‖	90VP	....
Shear across the grain	Yellow pine†	1 000	....	500LYP	1 20
	White pine	500	....	350	80
	Spruce‡	500	....	350	80
	Hemlock	600	....	350	...
	Douglas fir	1 000	....	....	1 00
	Oak	1 000	....	720	1 20
	Locust	......	....	....	....
	....	......	....	400VP	....

* Stresses named by N. Y. are given in the 1917 Building Code of the Boroug Ianhattan. Exception: Dist. of Columbia omits hemlock, omits chestnut in s cross grain and puts spruce and Virginia pine under one caption; Cincinnati m aption of white pine and spruce, with N. Y. white-pine values, and gives 270 for heml or shear across grain. † Chicago, "Douglas fir and long-leaf yellow pine." ‡Chic o values for spruce; spruce-values apply to Norway pine. ‖ Chicago, values g or short-leaf yellow pine, SLYP. § Baltimore, LLYP is long-leaf yellow pine: r VP, N. Carolina or Virginia pine. ¶ Boston, yellow pine is "yellow pine (long-le

10. Ultimate Unit Stresses for Woods

The Average Ultimate Unit Stresses for the CONIFEROUS or SOFTWOODS and for the BROAD-LEAVED or HARDWOODS, together with the AVERAGE WEIGHTS of the woods per cubic foot are given in Tables XVIII and XIX. The values given are compiled from many tests on numerous species of timber. In regard to the range of values for the same kind of wood, it may be stated that the higher values are for specimens which contained a percentage of water varying from 15 to 20%; and that tests on laboratory specimens showed greater strength than the actual pieces used in construction. The WEIGHTS PER CUBIC FOOT are averages of the weights of many specimens tested and agree generally with average values given in other tables of weights of materials.

Table XVIII.* Average Ultimate Unit Stresses for the Coniferous or Softwoods, in Pounds per Square Inch

Kind of wood	Weight in lb per cu ft, dry	Tension	Compression		Bending (modulus of rupture)	Shear	
			With the grain	Across the grain		With the grain	Acros[illegible] the grain
Cedar (white).......	19.72 to 20.70	8 000 to 11 400	4 000 to 6 000	700	5 000	400	1 30[illegible] to 1 51[illegible]
Cedar (red).........	23.66	8 000	4 000 to 7 000	700	5 000	......	1 50[illegible]
Cypress............	29.80	4 000 to 6 000	4 000 to 8 000	700 to 800	5 000 to 11 700	500	.....
Hemlock...........	26.42 to 32.29	6 000 to 8 700	4 000 to 7 420	600 to 700	3 500	350	2 50[illegible] to 2 75[illegible]
Pine (white)........	25.55	3 000 to 12 000	3 000 to 6 650	700 to 1 000	4 000 to 10 000	225 to 423	2 48[illegible]
Pine (red), (Norway pine)............	30.25	5 000 to 13 000	6 000 to 8 000	800 to 1 000	5 000 to 12 300	500	.....
Pine (yellow), (long-leaf)............	43.62	6 000 to 13 000	5 000 to 9 500	1 000 to 1 400	7 000 to 14 200	300 to 700	4 34[illegible] to 5 00[illegible]
Pine (yellow), (short-leaf)............	38.40	5 000 to 10 000	4 000 to 9 000	900 to 1 000	6 000 to 12 400	400 to 700	4 00[illegible] to 5 00[illegible]
Douglas fir (Oregon pine)............	32.14	9 000 to 14 000	4 880 to 9 800	800 to 1 200	6 500 to 12 100	500 to 600	.....
Redwood (California)	26.23	7 000 to 10 853	3 000 to 4 000	800	4 500	400	......
Spruce (black)......	28.57	5 000 to 19 500	4 000 to 7 850	700	4 000 to 12 000	250 to 400	3 25[illegible]
Spruce (white)......	25.25	5 000 to 19 500	4 000 to 7 850	700	4 000 to 12 000	250 to 400	3 25[illegible]

* The higher values of tensile and compressive strengths are for " dry " or " se[illegible]oned " timber containing from 10 to 15% of water. For safe fiber-stresses f[illegible] lexure, see Table I, page 557.

Table XIX.* Average Ultimate Unit Stresses for the Broad-Leaved or Hardwoods, in Pounds per Square Inch

Kinds of wood	Weight in lb per cu ft, dry	Tension	Compression		Bending (modulus of rupture)	Shear	
			With the grain	Across the grain		With the grain	Across the grain
Ash (white)	40.77	11 000 to 17 000	4 000 to 9 000	1 900	6 300 to 14 200	450 to 1 100	6 280
Ash (red)	38.96	......	6 800	......	......	......	......
Ash (green)	44.35	......	8 000 to 9 800	1 700	5 100 to 16 000	1 000	6 280
Chestnut	41.00	9 000 to 13 000	5 000	900	5 000	600	1 500
Elm (white)	45.26	8 000 to 13 000	6 000 to 10 000	1 200	7 300 to 13 600	800	......
Gum	36.83	15 000 to 18 000	5 600 to 8 500	1 400	6 000 to 12 700	800	5 890
Hickory	46.16 to 52.17	12 800 to 18 000	7 000 to 10 000	2 700 to 3 200	5 400 to 24 300	1 000 to 1 200	6 000 to 7 800
Locust	45.70	10 500 to 24 800	7 000 to 11 700	......	......	......	7 176
Lignum-vitæ	77.12	11 000	8 800	......	......	......	......
Maple (hard)	43.08	8 000 to 10 000	7 000 to 9 940	......	......	......	6 355
Maple (white)	32.84	8 000 to 10 000	6 000 to 7 500	1 700 to 1 900	......	399 to 537	6 355
Mahogany (Central America)	35.00	2 300 to 17 900	6 000	......	10 800	......	......
Oak (white)	46.35	10 000 to 19 500	4 500 to 11 300	2 000	6 000	750 to 1 000	4 425
Oak (chestnut)	53.63	10 000	7 500	......	......	......	......
Oak (live)	59 21	13 000	9 000	......	......	......	8 480
Oak (red and black)	40.75	10 000	4 000 to 8 500	2 300	9 100 to 15 400	1 100	......
Poplar (whitewood)	30.00	7 000	4 000 to 5 700	......	......	......	4 418
Walnut (white) (butternut)	25.46	......	5 000 to 6 800	......	......	......	2 830
Walnut (black)	38.11	12 000	7 500	......	......	......	4 728

* The higher values of the tensile and compressive strengths are for "dry" or "seasoned" timber containing from 10 to 15% of water. For safe fiber-stresses for use, see Table I, page 557.

CHAPTER XVII

STRENGTH OF BUILT-UP, FLITCHED AND TRUSS[E]D WOODEN GIRDERS

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1. Built-Up Wooden Girders

Built-Up Wooden Beams. Wooden beams or girders built up of pla[nks] spiked or bolted together side by side, will generally be somewhat stron[ger] than solid girders of the same dimensions, because the planks will be be[tter] seasoned and freer from check-cracks and other defects. For beams or gir[ders] 10 in or less in depth, spikes will usually be sufficient to bind the planks toget[her] but for deeper beams, bolts should be used in addition to the spikes, to prev[ent] the planks from separating and the outer planks from warping or curling a[way] from the others.

Bolts. Two bolts should be placed at each end of the beam and every [...] feet of its length.

Lengths of Planks. When a beam is built up in this way each pl[ank] should extend the full length of the beam. In a CONTINUOUS BEAM, the pla[nks] should break joints over the supports. The planks of BUILT-UP BEAMS sh[ould] always be set on edge, never flatwise.

Compound Wooden Beams. It is sometimes necessary to use a woo[den] beam for a longer span or greater load than is safe for the deepest SINGLE B[EAM] that can be obtained, or for a beam built up of planks. In such cases COMPO[UND] WOODEN BEAMS may be used.

Definition. By a COMPOUND WOODEN BEAM or GIRDER is meant a b[eam] built up by placing two or more single beams over another one, with the [...] of having them act [...] SINGLE BEAM having [...] depth of the comb[ined] beams.

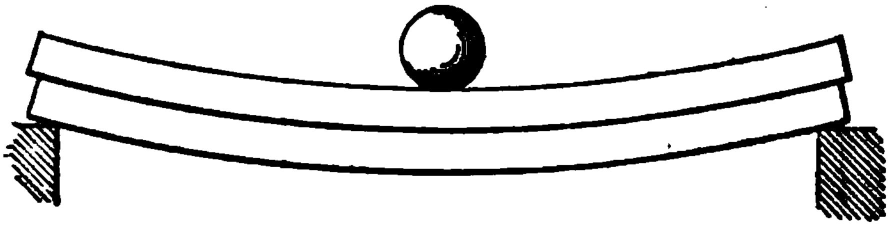


Fig. 1. Two Simple Wooden Beams, One Over the Other, Loaded in Middle

Strength of Co[m]pound Beams. If 10 by 10-in beams [...] placed one on top of [the] other, and the upper loaded at the middle, the beams would act as two separate beams (Fig. 1) [...] their combined strength would be no greater than if the two beams were pl[aced] side by side. If, however, the two beams can be joined so that the fibe[rs of] the lower beam will be extended as much as would be the case in a si[ngle] beam of the same depth, or, in other words so that the two beams will [not] slip on each other, the COMPOUND BEAM will have four times the strengt[h of] the SINGLE BEAM.

Tests of Compound Beams. Various attempts have been made to [...] beams thus placed so as to prevent the two parts slipping on each other.

ring the years 1896–7, Edgar Kidwell, of the Michigan College of Mines, ade an extended series of tests of the efficiency of COMPOUND BEAMS of differ-t patterns. From these tests much valuable data was obtained. A full cription of the tests, accompanied by the conclusions of the author, and the s and data for proportioning the bolts and keys, of KEYED BEAMS, is pub-ed in the Trans. Am. Soc. M. E., vol. 27.

Simple Form of Compound Beam. A form of COMPOUND BEAM, some-es used in American building-construction, is shown in Fig. 2, diagonal rds in opposite directions being nailed to each side of the two timbers to vent their slipping on each other. T. M. Clark, in his Building Superin-dence, advocates this as one of the best forms of compound beams, and

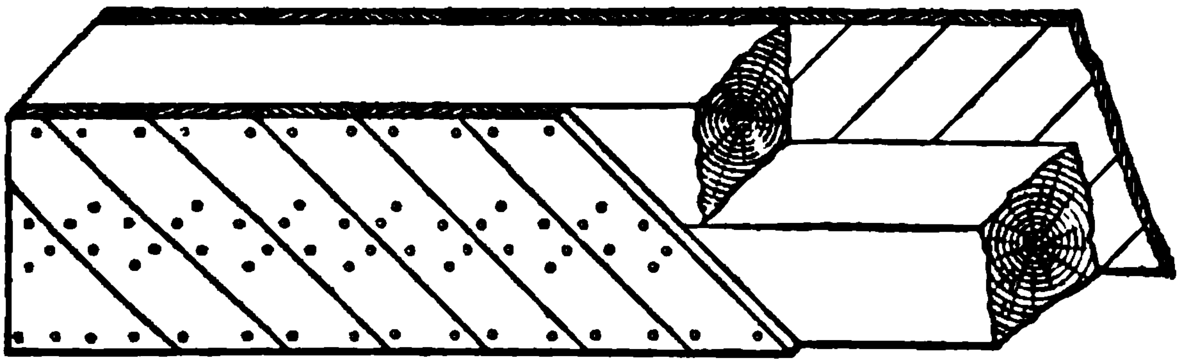


Fig. 2. Simple Form of Compound Wooden Beam

es its EFFICIENCY at about 95% of that of a solid beam of the same depth. fessor Kidwell made nine tests of this type of beam. In six of the beams ratio of span to depth was as 12 to 1, and in three of the beams, as 24 to 1. shorter beams gave an average EFFICIENCY, without much variation, of %, and the longer beams an EFFICIENCY of 80.7%.

It was found that the beams failed by the splitting of the diagonal pieces or drawing of the nails; "in every case, long before the beam broke, the struts open or the nails were partly drawn out or bent over in the wood, thereby itting the component beams to slide on each other." When built with onal boards, 1¼ in thick, nailed with tenpenny nails, as in Fig. 2, the KING STRENGTH of such a beam may be taken at 65% of the strength of

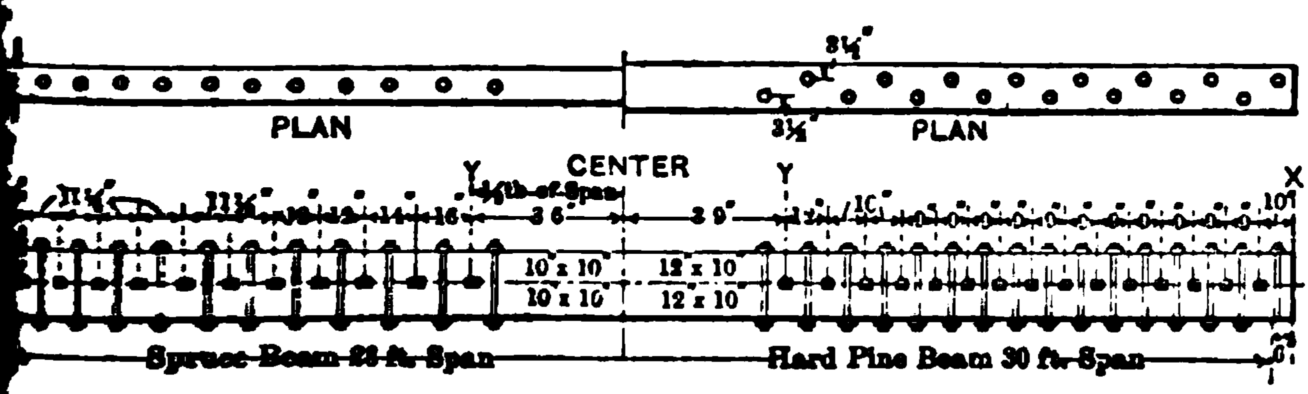


Fig. 3. Compound Keyed and Bolted Wooden Beams

d beam of the same depth and of a breadth equal to the breadth of the ers. The DEFLECTION of the beam, however, will be about double that of d beam of the same size, and on that account this type of beam is not recommended for supporting floors with plastered ceilings or for carrying ered partitions.

eyed Beams. Professor Kidwell tested, also, several types of KEYED , and found that a compound beam keyed and bolted together, as shown g. 3, is the most efficient form that it is practical to build.

It was found that with oak keys it was possible to obtain an EFFICIENCY spruce beams of 95%, while the DEFLECTION varied from 20 to 25% more t would be expected in a solid beam.

Cast-Iron Keys. By using CAST-IRON KEYS the deflection was found t but little, if any, greater than for a solid beam.

Shape of Keys. The keys must be wedge-shaped, as shown in Fig. 4 that they can be driven tightly against the end-wood.

Efficiency of Keyed Beams. Professor Kidwell recommends that ordinary purposes an EFFICIENCY of 75% be allowed when oak keys are and of 80% when the keys are of cast iron. The width of an oak key sh be twice its height. Numerous small keys closely spaced gave better re than fewer large keys. In his report, Professor Kidwell gives formulas, for the number and spacing of the keys.

Keys, Bolts and Washers for Compound Beams. As compound be when used, are generally built up of 8, 10, 12 or 14-in timbers, Mr. Kic some years ago, prepared a table giving the sizes of keys, the number requ on each side of the middle of the span, their minimum spacing and the siz the bolts and washers to be used for such beams of from 20 to 36-ft sp He noted that the MAXIMUM SAFE LOADS for such beams should be 75% o loads computed by Formula (10), page 630, for a beam supported at both e and loaded with a uniformly distributed load.

Table I. Keys, Bolts and Washers for Compound, Keyed Wooden Bea

Size of beams	Size of keys	Bolts	Washers	Number of keys each si of center line			
				White pine	Spruce	Doug-las fir	L[illegible]
16-in beams	1½ by 3 -in oak keys	¾-in	3 -in	7	8	11	
20-in beams	1½ by 3 -in oak keys	¾-in	3 -in	9	11	13	
24-in beams	2 by 4 -in oak keys	⅞-in	3½-in	8	9	12	
28-in beams	2¼ by 4½-in oak keys	⅞-in	3½-in	9	10	12	

Size of keys	Bolts	Washers	Minimum spacing of ke			
1½ by 3 -in oak keys	¾-in	3-in	11¼-in	11¼-in	9 -in	9
2 by 4 -in oak keys	⅞-in	3-in	15 -in	15 -in	11½-in	11
2¼ by 4½-in oak keys	⅞-in	3-in	17 -in	17 -in	13 -in	13

The Breadth or Thickness of Compound Beams should be not less two-fifths of the depth.

The Number of Keys required is not affected by the length or bread the beam, if the beam is figured for the full safe load.

In Spacing the Keys (Figs. 3 and 4) they should not be closer tha minimum spacing given in Table I. For beams loaded at the middle, the ing of the keys should be uniform from X to Y, Fig. 3, Y being one-eight span from the center line. If the distance between the keys, center to

works out less than the minimum spacing, the safe load should be cor-ndingly reduced or the thickness of the beam increased.

Beams Uniformly Loaded, the first four or five keys from the end ld be spaced for minimum spacing, and the spacing of the remaining keys ased toward the point *Y*. When the ratio of depth to span is greater than 16, the inner key may be a little more than one-eighth the span from the er line, for distributed loads. Fig. 3 shows the proper spacing for a 20-in e beam of 28-ft span and for a long-leaf yellow pine beam of 30-ft span; the tabulation below gives the proper spacing of keys for spruce beams of

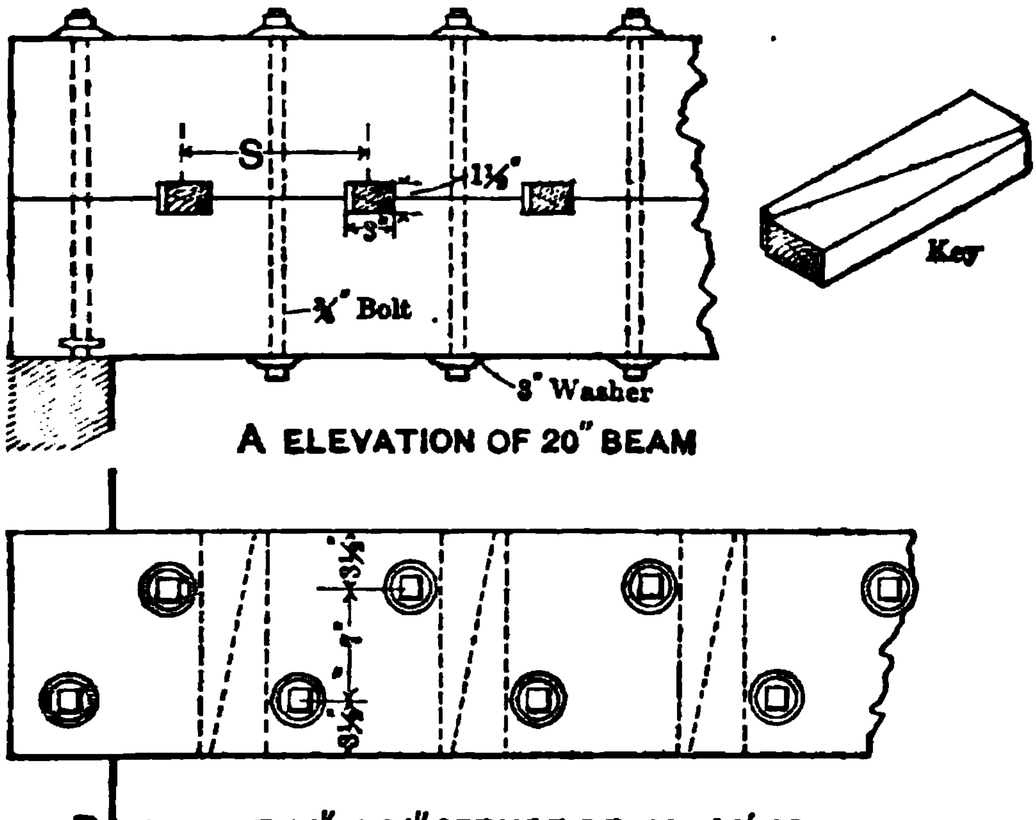


Fig. 4. Details of Keyed and Bolted Wooden Beam

r spans, figured from the end of the beam in each case. For other woods spans the spacing should be made as near like these as the fixed condi- will permit. Four examples of spacing are given below. The sizes of and washers to be used are given in Table I. If the beam is not over wide, the bolts may be arranged as for the spruce beam (Fig. 3); if 12 in or over, the bolts should be staggered as shown for the hard-pine beam. very wide beam the bolts might be spaced as in detail *B*, Fig. 4.

cing of keys in inches for spruce beams, commencing at end, for uniformly buted loads:

in spruce beam, 32-ft span,	10,	12,	12,	16,	19, 24, 32
in spruce beam, 32-ft span,	10,	11½,	11½,	11½,	12, 12, 12, 13, 15, 18, 24
in spruce beam, 36-ft span,	13,	15,	15,	15,	15, 16, 18, 20, 30
in spruce beam, 36-ft span,	15,	17,	17,	17,	17, 17, 17, 17, 17, 17

2. Flitched Beams or Flitch-Plate Girders

ch-Plate Beams (Fig. 5) were at one time much used, but with the t prices of steel it is cheaper and better to use steel beams.

following explanation and formulas are given, however, for the benefit e who might have occasion to use a beam of this kind. It has been in practice that the thickness of the wood should be sixteen times the ss of the steel. As the steel is so much STIFFER than the wood, we must

proportion the load on the wood so that the latter will bend as much steel plate bends: otherwise the whole load might be thrown on the steel The MODULUS OF ELASTICITY of steel is about twenty times that of lo yellow pine; so that a beam of this wood, 1 in wide, will bend twenty as much as a plate of steel of the size and under the same load. if we want this beam to bend much as the steel plate, we mus only one-twentieth the load on the wooden beam is sixteen tim thick as the steel plate, we should put sixteen-twentieths of its safe load or, what amounts to the same thing, use a constant only four-fifths strength of the wood.

Fig. 5. Flitch-plate Girder

Formulas for Flitch-Plate Girders. On this basis the following for have been derived for the strength of FLITCH-PLATE GIRDERS, in which the ness of the wood is sixteen times the breadth of the steel, approximately:

Let d = depth of beam in inches
b = total thickness of wood in inches
l = clear span in feet
t = thickness of steel plate in inches

$$A'^{*} = \begin{cases} 53.6 \text{ for long-leaf yellow pine} \\ 45 \text{ for Douglas fir} \\ 31 \text{ for spruce} \end{cases}$$

P = total load at middle in pounds
W = distributed load in pounds

Then, for beams supported at both ends,

$$\text{Safe load at middle in pounds} = \frac{d^2}{l}(A'b + 889\,t)$$

$$\text{Safe distributed load in pounds} = \frac{2\,d^2}{l}(A'b + 889\,t)$$

$$\text{For distributed load,} \qquad d = \sqrt{\frac{Wl}{2\,A'b + 1778\,t}}$$

$$\text{For load at middle,} \qquad d = \sqrt{\frac{Pl}{A'b + 889\,t}}$$

The bolts should be ¾ in in diameter, and spaced 2 ft on centers. end should have two bolts, as in Fig. 5.

Example. What is the safe load, uniformly distributed, for a girder com of three 4 by 14-in Douglas-fir timbers and two ⅜ by 14-in flitch-plates, span of 25 ft?

Solution. By Formula (2),

$$\text{Safe load} = \frac{2 \times 196}{25}(45 \times 12 + 889 \times 3/4) = 18\,922 \text{ lb}$$

3. Trussed Beams and Girders

Use of Trussed Beams and Girders. Whenever we wish to supp floor upon girders having a span of more than 30 ft, we must use a TR GIRDER, a riveted STEEL-PLATE GIRDER, or two or more STEEL BEAMS.

* For commercially seasoned timber and for ideal conditions these values may i about 30%.

me circumstances and in some parts of the country it may be cheaper or re convenient to use a large wooden girder, and truss it, as in Figs. 6, 7, 8 or 9.

Depth of Trussed Girder. For all these forms it is desirable to give the ders as much depth as the conditions allow; as, the deeper the girder, the aller the stresses in the pieces.

In the Single-Strut Trussed Girder, we either have two beams, and one d which runs up between them at the ends, or three beams, and two rods ning up between the beams in the same way. The beams should be in one ntinuous length for the whole span, if they can be obtained in that length. e requisite dimensions of the tie-rod, struts and beams, in any given case, st be determined by first finding the stresses developed in these pieces, and en the areas of cross-sections required to resist these stresses.

For a Single-Strut Truss (Fig. 6), the stresses in the pieces may be deter-ned by the following formulas:

r a Distributed Load *W* Over the Whole Girder (Fig. 6)

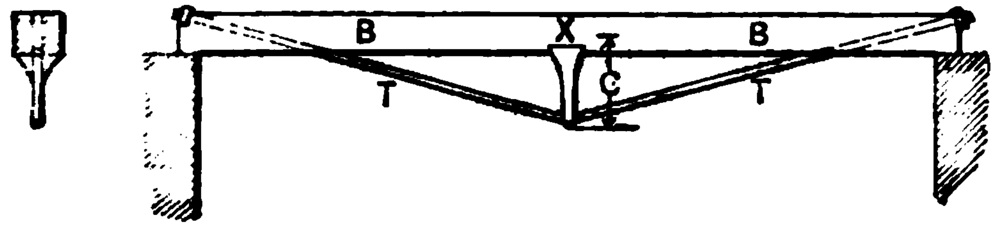


Fig. 6. Trussed Wooden Girder. One Vertical Strut

$$\text{Tension in } T = \frac{W}{2} \times \frac{\text{length of } T}{\text{length of } C} \qquad (5)$$

$$\text{Compression in } C = \tfrac{5}{8} W. \quad \text{(See Note.)} \qquad (6)$$

$$\text{Compression in } B = \frac{W}{2} \times \frac{\text{length of } B}{\text{length of } C} \qquad (7)$$

te. When the beam B is in one piece, the full length of span. If B is ted over the strut then compression in C or tension in $R = \tfrac{1}{2} W$.

r a Concentrated Load *P* Over *C* (Fig. 6)

$$\text{Tension in } T = \frac{P}{2} \times \frac{\text{length of } T}{\text{length of } C} \qquad (8)$$

$$\text{Compression in } C = P$$

$$\text{Compression in } B = \frac{P}{2} \times \frac{\text{length of } B}{\text{length of } C} \qquad (9)$$

r a Girder Trussed as in (Fig. 7), Under a Distributed Load *W* Over the le Girder

$$\text{Compression in } S = \frac{W}{2} \times \frac{\text{length of } S}{\text{length of } R} \qquad (10)$$

$$\text{Tension in } R = \tfrac{5}{8} W. \quad \text{(See Note.)}$$

$$\text{Tension in } B = \frac{W}{2} \times \frac{\text{length of } B}{\text{length of } R} \qquad (11)$$

te. When the beam B is in one piece, the full length of span. If B is d over the strut then compression in C or tension in $R = \tfrac{1}{2} W$.

For a Concentrated Load, P at the Middle (Fig. 7)

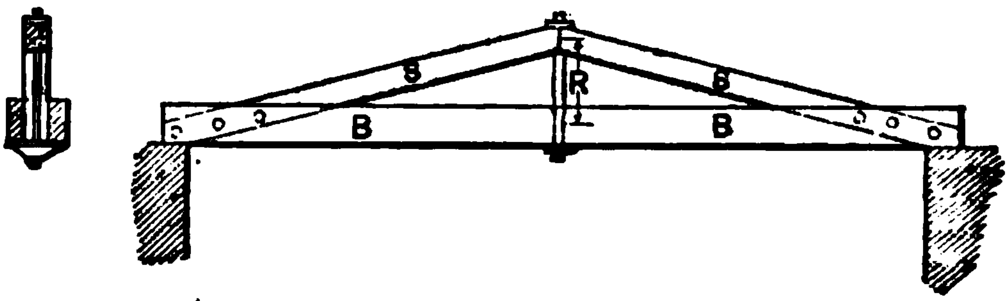


Fig. 7. Trussed Wooden Girder. One Vertical Tie

$$\text{Compression in } S = \frac{P}{2} \times \frac{\text{length of } S}{\text{length of } R}$$

$$\text{Tension in } R = P$$

$$\text{Tension in } B = \frac{P}{2} \times \frac{\text{length of } B}{\text{length of } R}$$

For a Double-Strut Trussed Beam (Fig. 8) with a Distributed Load W the Whole Girder (Beam B Divided into Three Equal Spans)

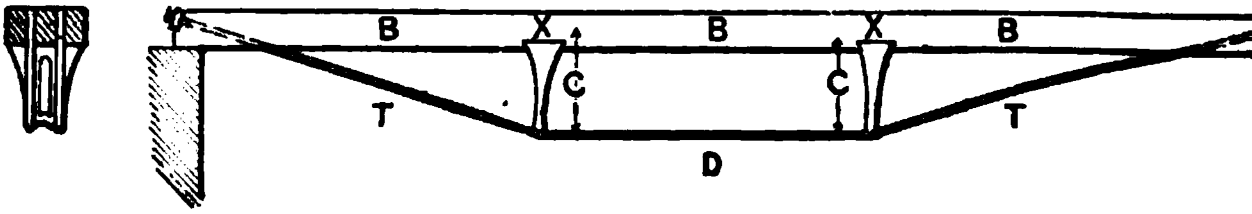


Fig. 8. Trussed Wooden Girder. Two Vertical Struts

$$\text{Tension in } T = \frac{W}{3} \times \frac{\text{length of } T}{\text{length of } C}$$

$$\text{Compression in } C = \frac{W}{3}$$

$$\text{Compression in } B \text{ or tension in } D = \frac{W}{3} \times \frac{\text{length of } B}{\text{length of } C}$$

For a Concentrated Load P Over Each of the Struts C (Fig. 8)

$$\text{Tension in } T = P \times \frac{\text{length of } T}{\text{length of } C}$$

$$\text{Compression in } C = P$$

$$\text{Compression in } B \text{ or tension in } D = P \times \frac{\text{length of } B}{\text{length of } C}$$

For a Girder Trussed as in Fig. 9, and Under a Distributed Load W the Whole Girder (Beam B Divided into Three Equal Spans)

$$\text{Compression in } S = \frac{W}{3} \times \frac{\text{length of } S}{\text{length of } R}$$

$$\text{Tension in } R = \frac{W}{3}$$

$$\text{Tension in } B \text{ or compression in } D = \frac{W}{3} \times \frac{\text{length of } B}{\text{length of } R}$$

For Concentrated Loads P Applied at Joints 2 and 3 (Fig. 9)

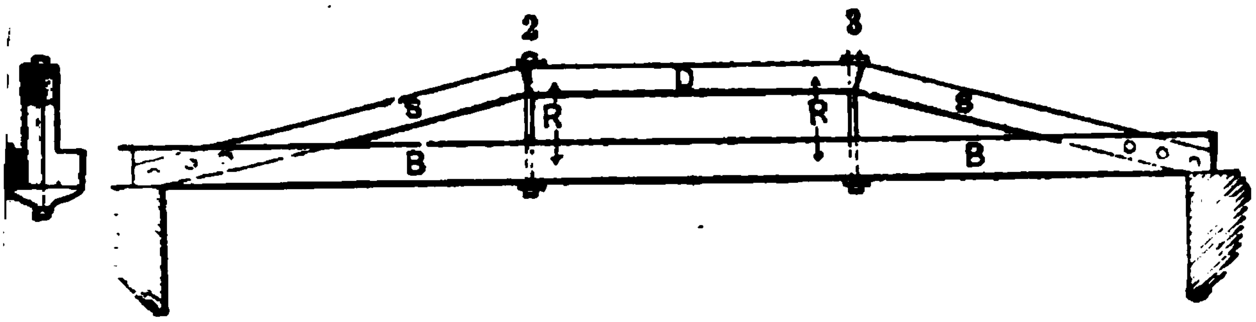


Fig. 9. Trussed Wooden Girder. Two Vertical Ties

$$\text{Compression in } S = P \times \frac{\text{length of } S}{\text{length of } R} \tag{20}$$

$$\text{Tension in } R \quad = P$$

$$\text{Tension in } B \text{ or compression in } D = P \times \frac{\text{length of } B}{\text{length of } R} \tag{21}$$

Trusses constructed as shown in Figs. 8 and 9 should be divided so that the rods R, or the struts C, will divide the lengths of the girder into three equal or nearly equal parts. The lengths of the pieces T, C, B, R, S, etc. should be measured ON THE AXIAL LINES of the pieces. Thus, the length of R should be measured from the CENTER LINE OR AXIS of the tie-beam B to the CENTER LINE or axis of the strut D; and the length of C should be measured from the AXIS of the rod to the AXIS of the strut-beam B.

After determining the stresses in the pieces by these formulas, we may compute the areas of the cross-sections by the following rules:

$$\text{Area of cross-section of a short strut} = \frac{\text{compression in strut}}{S_c} \tag{22}$$

in which S_c for cast iron may be taken at from 13 000 or 14 000 lb per sq in, and for wood as given in Table XVI, page 647.

The size of the long strut D (Fig. 9) should be determined by means of Tables 451 and 452 for wooden columns, Chapter XIV.

The diameters of the tie-rods may be obtained from Table II, page 388.

For the beam B (Figs. 8 and 9) when the load is distributed, we must compute the necessary area of cross-section as a STRUT (Fig. 8) or a TIE (Fig. 9), and also the area of its cross-section, as a BEAM, required to support its load, and use a beam with a section equal to the sum of the two sections thus obtained.

$$\left.\begin{array}{c}\text{Area of cross-section of } B \text{ to resist}\\ \text{tension or compression}\end{array}\right\} = \frac{\text{tension}}{S_t} \text{ or } \frac{\text{compression}}{S_c} \tag{23}$$

For the trusses shown in Figs. 6 and 7, with distributed loads,

$$\text{Breadth of } B \text{ (as a beam)} = \frac{W \times l}{4 \times d^2 \times A} \tag{24}$$

For the trusses shown in Figs. 8 and 9, with distributed loads,

$$\text{Breadth of } B \text{ (as a beam)} = \frac{W \times l}{6 \times d^2 \times A} \tag{25}$$

(Compare Equation (24) and (25) with Equation (11), page 630.)

W denotes the total distributed load in pounds on the girder, and l the length in feet of one section of the beam. When the loads are concentrated over the struts C (Fig. 8) or at the joints R (Fig. 9) then there will be no TRANSVERSE

STRESS on the beams B, and they need be proportioned for the COMPRES or TENSILE STRESS, only, as the case may be.

In Formulas (23), (24) and (25), for S_c and S_t, substitute the values safe unit stresses for compression, Table XVI, page 647, and for tension, T II, page 388, and for A substitute the values recommended in Tables II XVI, pages 628 and 647.

Illustrative Examples. To illustrate the method of computing the di sions of the different parts of girders of this kind, two examples are given.

Example 1. It is required to design a trussed girder of the form show Fig. 6, for a span of 30 ft. The girders are to be 12 ft on centers, and a carry a floor loaded with 100 lb per sq ft. The girder consists of three s beams B, side by side, and two rods. We can allow the rod T to come two below the beams B, and we will assume that the depth of the beams B be 12 in; then the length of C, measured from the center line of the beam, be 30 in. The length of B is 15 ft, and by computation, or by scaling, we the length of T to be 15 ft 2½ in.

Solution. The total load on the girder equals 100 lb multiplied by the multiplied by the distance of the girders on centers, or, $100 \times 30 \times 12 = 36$ 0
From Formula (5),

$$\text{Tension in } T = \frac{36\,000}{2} \times \frac{182\tfrac{1}{2}\text{ in}}{30\text{ in}} = 109\,500\text{ lb}$$

or 54 750 lb on each of the two rods. For such a large stress it is best to the ends of the rods, and allowing 16 000 lb per sq in for steel rods, we find Table II, Chapter XI, that we must use two 2⅛-in steel rods.

The strut-beam we will make of long-leaf yellow pine. From Formula (find the compressive stress in $B = (36\,000/2) \times (180/30) = 108\,000$ lb. As w to use three beams side by side, there will be 36 000 lb compression in each b

To resist the compression there is required an area of 36 000/1 000 or 36 s which is equal to 3 by 12 in.

From Formula (24) we find the total breadth required to resist the trans stress $= \dfrac{36\,000 \times 15}{4 \times 144 \times 67} = 14$ in; or each beam must be 4⅔ by 12 in in secti resist the transverse stress, and 3 by 12 in to resist the compressive s Consequently each beam must be 7⅔ by 12 in in cross-section.

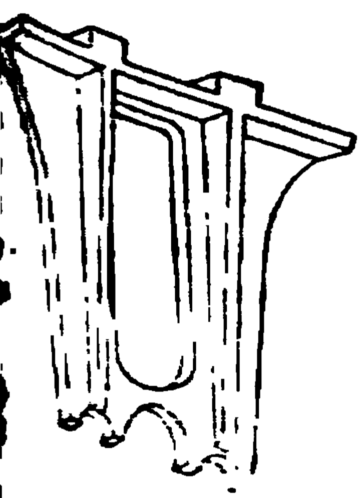
As this would make the girder very wide, 27¼ in, we will use beams deep, increasing the depth of the girder 1 in, so that the height on center still be 30 in.

The area required to resist the compressive stress will be the same as b 36 in, but as the beam is 14 in deep the breadth will be only 2.57 in.

The total breadth to resist the transverse stress will be $\dfrac{36\,000 \times 15}{4 \times 196 \times 67} = 10.$ or 3.43 in for each beam. The total breadth for each beam will therefo 6 in. A beam with a cross-section of 6 by 14 in will meet the requiren The total width of the girder will then be 22¼ in. The load on $C =$ ⅝ 22 500 lb, or 11 250 lb over each rod. The theoretical sectional area in s inches necessary to resist this load = 11 250/13 000 for cast iron and 11 250/ for oak. As the struts must be the full width of the girder, however, i be necessary to make the sectional area much greater than the theoretic quirements. If made of cast iron the strut should be of the shape sho Fig. 10, and if of oak, of the shape shown in Fig. 11. The cast-iron stru be the best, but an oak strut will answer.

mple 2. It is required to support a floor over a lecture-room 40 ft wide, neans of trussed girders; and as the room above is to be used for electrical poses, it is desired to have a truss with very little iron in it. It is decided, efore, to use a truss such as is shown in Fig. 9.

lution. Where the girders rest on the wall, there will be brick pilasters ing a projection of 6 in, which will make the span of the truss 39 ft, and rods *RR* will be placed so as to divide the tie- into three equal spans of 13 ft each. The tie-beam *B* will consist of two long-leaf yellow pine beams, with the struts *S* coming between them. There are two rods, instead of one, at *RR*, coming down on each side of the struts *S*, and passing through iron castings below the beams *B*, and forming supports for them. The height of the truss from center to center of timbers must be limited to 18 in. The trusses are 8 ft on centers.



. 10. Cast-iron Strut for Two Tie-rods

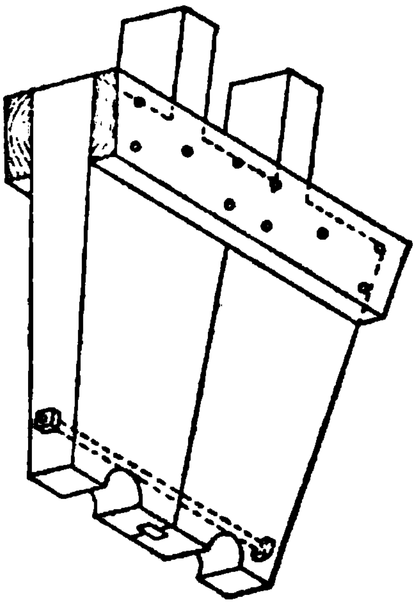


Fig. 11. Wooden Strut for Two Tie-rods

he total floor-area supported by one girder is 8 by 39 ft, or 312 sq ft. The iest load to which the floor will be subjected is the weight of the people the room, for which 75 lb per sq ft is an ample allowance; and the weight he floor itself is about 100 lb per sq ft. This makes the total weight liable ome on one girder 31 200 lb.

$$\text{Compression in } S\text{, from Formula (18)} = \frac{W}{3} \times \frac{157 \text{ in}}{18 \text{ in}} = 90\,700 \text{ lb}$$

$$\text{Tension in one pair of rods} = \frac{W}{3} = 10\,400 \text{ lb}$$

$$\text{Tension in } B \text{ or compression in } D = \frac{W}{3} \times \frac{156 \text{ in}}{18 \text{ in}} = 90\,130 \text{ lb}$$

the unsupported length of *D* is greater than that of *S*, a beam that will t the compression in *D* will be ample for *S*. We find from Table II, Chap. XIV, that it will require a post 10 by 12 in in cross-section to resist the com- sion in *D*, which is 13 ft in length. The tension in each rod is only 5 200 lb; as the rods must support a larger washer at the bottom, they are made in diameter, not upset. The tension in each of the beams *B* is 45 065 lb. divided by 1 200, the safe unit tensional stress for long-leaf yellow pine 37.6 sq in, or about 2¾ by 14 in.

he total breadth of the tie-beam to resist the transverse load is found from nula (25). Assuming 14 in for the depth of *B*

$$\text{Breadth of } B = \frac{31\,200 \times 13}{6 \times 196 \times 67} = 5.15 \text{ in, or about } 2\tfrac{1}{2} \text{ in for each beam}$$

he breadth of each tie-beam must therefore be 2¾ in + 2½ in = 5¼ in. ce the tie-beams must be 5 by 14 in in section. The girder, therefore, must built with 10 by 14-in strut-beams and two 5 by 14-in tie-beams, each 42 ft . The 1-in rods may be cut ½ in into the strut-beams, and ½ in into the tie-

beams, so that the latter will come close against the struts S. The thrus the strut S is equal to its compressive stress, and a connection between the beams and the struts must be designed that will resist this thrust, which in case is 90 700 lb. As the inclination of the strut is very slight, there is an room for bolts. It is best to use bolts which are at least 1½ in in diame As they are in double shear, the resistance to shearing of one bolt is 35 34 (See Table IX, page 431.) Steel bolts are used.

The bearing area of a 1½-in bolt in a timber 10 in wide is 15 in. For bearing resistance of long-leaf yellow pine, we may allow 1 400 lb per s (Table XVI, page 647), which will give 21 000 lb as the bearing resistanc one 1½-in steel bolt. As the force to be resisted is 90 700 lb, it will req five 1½-in steel bolts to sustain this bearing pressure, the resistance to shea being greater than this stress.

The number of bolts required to resist the bending moment must now determined. The total bending moment to be resisted (see page 434, Cl ter XII) = 90 700 times the distance, in inches, between the centers of the beams divided by 12, or $90\,700 \times {}^{15}\!/_{12} = 113\,375$ in-lb.

From Table IX, page 431, we find that the maximum bending mom at a fiber-stress of 20 000 lb per sq in, for a 1½-in steel bolt is 6 630 i Hence it will require seventeen 1½-in bolts to resist the thrust in S witl bending the bolts. As it is impracticable to put in so many bolts, larger must be used. For a 2½-in steel bolt, the maximum bending momer 30 700 in-lb (Table IX, page 431), and four times this is 122 800 i hence four 2½-in steel bolts will be sufficient to resist the bending stress, also the shearing and bearing-stresses. It will be seen from this example it is much more difficult and expensive to make satisfactory end-joint girders trussed as in Figs. 7 and 9 than it is for the single and double-s trusses like those shown in Figs. 6 and 8. The latter forms are to be prefe when the conditions will admit of their use.

These four cases of TRUSSED GIRDERS are but special examples of tru The stresses in them may also be determined by the methods explaine Chapter XXVII; and where the divisions of the girder cannot be made form, the stresses should be computed by the general methods there explai

CHAPTER XVIII

STIFFNESS AND DEFLECTION OF BEAMS

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1. General Principles of the Deflection of Beams*

Strength and Stiffness. In many structures it is necessary that beams and girders shall be not only STRONG enough but STIFF enough; that is, not only must RUPTURE be prevented, but the beams must not BEND so much as to appear unsightly, or to crack the ceiling. Therefore, in many cases, DEFLECTION, rather than absolute STRENGTH, may become the governing consideration in determining the size of a beam. Unfortunately, there is no method at present of combining the two calculations for STRENGTH and for STIFFNESS in one. A beam properly proportioned for STRENGTH will not bend enough to stress the fibers beyond the ELASTIC LIMIT, but it may in some cases bend more than a due regard for appearances will justify. The distance that a beam bends under a given load is called its DEFLECTION, and its resistance to deflection is called its STIFFNESS.

A General Formula for the Maximum Deflection of any beam under a concentrated or uniformly distributed load not stressed beyond the ELASTIC LIMIT is:

$$\text{Deflection in inches} = \frac{\text{load in pounds} \times \text{cube of span in inches} \times c}{\text{modulus of elasticity} \times \text{moment of inertia}} \qquad (1)$$

The values of c are as follows:

For beam supported at both ends, loaded at the middle......	0.021
For beam supported at both ends, uniformly loaded.........	0.013
For cantilever beam, loaded at free end....................	0.333
For cantilever beam, uniformly loaded.....................	0.125

Deflection of Beam with Rectangular Section. By making the proper substitutions in Formula (1), the following formula for a RECTANGULAR beam SUPPORTED AT BOTH ENDS and LOADED AT THE MIDDLE may be derived:

$$\text{Deflection in inches} = \frac{\text{load in pounds} \times \text{cube of span in feet} \times 1\,728}{4 \times \text{breadth} \times \text{cube of depth} \times E} \qquad (2)$$

Modulus of Elasticity. From this formula the value of the MODULUS OF ELASTICITY, E, for different materials, has been calculated by accurately measuring the actual deflection of known beams under given loads applied at the middle and then substituting these known quantities in Formula (2).

Simple Formula for Deflection. Formula (2) may be simplified somewhat by representing $1\,728/4E$ by $1/F$, which gives the formula

$$\text{Deflection in inches} = \frac{W \times l^3}{b \times d^3 \times F\dagger} \qquad (3)$$

For a DISTRIBUTED LOAD the deflection will be five-eighths of this.

* See also, in Chapter XVI, formula on page 636 and Table XVI, page 647.

† The constant F corresponds to Hatfield's F, in his treatise on "Transverse Strains."

To Find the Load at the Middle that will cause a GIVEN DEFLECTI transpose Formula (2) so that the load becomes the left-hand member of equation. Thus:

$$\text{Load at middle in pounds} = \frac{4 \times \text{breadth} \times \text{cube of depth} \times \text{deflection in in} \times E}{\text{cube of span} \times 1\,728}$$

Limit of Deflection. In order that this formula may be of use in de mining the maximum load which may be placed upon a beam, the LIMIT OF DEFLECTION must in some way be fixed. This is generally done by making certain proportion of the span.

Allowable Deflection of Floor-Beams. Tredgold and other authori state that if a floor-beam deflects more than ONE-FORTIETH OF AN INCH EVERY FOOT OF SPAN, it is liable to crack the ceiling on the under side; and he this is the limit which is often set for the deflection of beams in first-c buildings.

Formulas for Deflection of Floor-Beams. If the length in feet divi by 40 is substituted for the deflection in inches, Formula (4) becomes

$$\text{Load at the middle} = \frac{\text{breadth} \times \text{cube of depth} \times e}{\text{square of span in feet}}$$

in which
$$e = \frac{E}{17\,280}$$

Most engineers and architects, however, think that ONE-THIRTIETH OF INCH PER FOOT OF SPAN, that is, 1/360 of the span, is not too much to allow the deflection of floor-beams, as a floor is seldom subjected to its full estim load, and then only for a short time.

Table I. Values of Constants for Stiffness or Deflection on Beams*

Material	E †	$F = \frac{E}{432}$	$e = \frac{E}{17\,280}$	$e_1 = \frac{E}{12\,9}$
Cast iron	15 000 000	34 722	862	1 157
Wrought iron	26 000 000	60 000	1 500	2 000
Steel	29 000 000	67 130	1 678	2 238
Ash	1 482 000	3 430	87	114
California redwood	700 000	1 620	40	54
Cedar	700 000	1 620	40	54
Chestnut	900 000	2 080	52	69
Cypress	900 000	2 080	52	69
Douglas fir	1 500 000	3 472	87	116
Hemlock	900 000	2 080	52	69
Long-leaf yellow pine	1 500 000	3 472	87	116
Maple	1 902 000	4 400	110	146
Norway pine	1 100 000	2 546	64	85
Short-leaf yellow pine	1 200 000	2 777	69	92
Spruce	1 200 000	2 777	69	92
White oak	1 500 000	3 472	87	116
White pine	1 000 000	2 315	58	77

E = modulus of elasticity, pounds per square inch; seasoned timber;
F = constant for deflection of beam, supported at both ends, loaded at middl
e = constant, allowing a deflection of one-fortieth of an inch per foot of span
e_1 = constant, allowing a deflection of one-thirtieth of an inch per foot of spa

* See, also, in Chapter XVI formula on page 636, and Table XVI, page 647.
† See Notes, page 637, regarding reductions in value for E, for unseasoned timb

If this ratio is adopted, the CONSTANT FOR DEFLECTION becomes

$$e_1 = \frac{E}{12\,960}$$

Constants for Stiffness or Deflection of Beams. In either of the above cases it is evident that the values used for E, F, e, or e_1, should be derived from tests on timbers of the same size and quality as those to be used. The values for the various woods given in the preceding table have been adopted by the editors after careful comparison with the results of numerous tests on large timbers and with values given by different authorities. The editors believe that they are perfectly reliable for first-class, seasoned, merchantable timber.

2. Formulas for Loads, Based Upon the Stiffness of Beams

Safe Loads for Limited Deflections for Rectangular Beams. Knowing the deflection caused by a load concentrated at the middle of a beam, and the RATIO OF OTHER DEFLECTIONS, caused by different modes of loading and supporting, formulas for Cases I to VIII, Figs. 1 to 8, considered under the strength of RECTANGULAR BEAMS (Chapter XVI), can be easily deduced. These cases, arranged in a different order, are:

For Beams Supported at Both Ends*

Load at the middle

$$\text{Safe load} = \frac{\text{breadth} \times \text{cube of depth} \times e_1}{\text{square of span}} \qquad (6)$$

$$\text{Breadth} = \frac{\text{load} \times \text{square of span}}{\text{cube of depth} \times e_1} \qquad (7)$$

Load at a point other than at the middle, ***m*** **and** ***n*** **being the segments into which the beam is divided**

$$\text{Safe load} = \frac{\text{breadth} \times \text{cube of depth} \times \text{square of span} \times e_1}{16 \times m^2 \times n^2} \qquad (8)$$

$$\text{Breadth} = \frac{\text{load} \times m^2 \times n^2 \times 16}{\text{cube of depth} \times \text{square of span} \times e_1} \qquad (9)$$

Load uniformly distributed

$$\text{Safe load} = \frac{8 \times \text{breadth} \times \text{cube of depth} \times e_1}{5 \times \text{square of span}} \qquad (10)\dagger$$

$$\text{Breadth} = \frac{5 \times \text{load} \times \text{square of span}}{8 \times \text{cube of depth} \times e_1} \qquad (11)$$

Inclined beam, loaded at the middle‡

$$\text{Safe load} = \frac{\text{breadth} \times \text{cube of depth} \times e_1}{\text{span} \times \text{horizontal distance between supports}} \qquad (12)$$

$$\text{Breadth} = \frac{\text{load} \times \text{span} \times \text{horizontal distance between supports}}{\text{cube of depth} \times e_1} \qquad (13)$$

* In Formulas (6) to (17) the breadth and depth are to be taken in inches, and the length or span in feet. The load is always in pounds.
The values given in either of the last two columns of Table I may be used for e or e_1, according to the degree of stiffness desired, but the values e_1 in the last column are ample under ordinary conditions.

† See, also, formula in Chapter XVI, page 636.

‡ Tredgold's Elements of Carpentry, page 65.

For Beams Fixed at One End, or Cantilever Beams

Load at the free end

$$\text{Safe load} = \frac{\text{breadth} \times \text{cube of depth} \times e_1}{16 \times \text{square of span}}$$

or,

$$\text{Breadth} = \frac{16 \times \text{load} \times \text{square of span}}{\text{cube of depth} \times e_1}$$

Load uniformly distributed

$$\text{Safe load} = \frac{\text{breadth} \times \text{cube of depth} \times e_1}{6 \times \text{square of span}}$$

or,

$$\text{Breadth} = \frac{6 \times \text{load} \times \text{square of span}}{\text{cube of depth} \times e_1}$$

Note. When the span in feet is less than the depth in inches, the be should not be calculated by the formulas for STIFFNESS, but by those for HO ZONTAL SHEAR. (See Chapter XVI, page 635.)

3. Relative Stiffness of Beams

Beam supported at both ends and loaded at the middle.........	1
Beam supported at both ends and uniformly loaded.............	8/
Beam fixed or restrained at both ends and loaded at the middle...	4
Beam fixed or restrained at both ends and uniformly loaded.....	8
Cantilever beam loaded at the free end........................	1/1
Cantilever beam uniformly loaded..............................	1/6

The Stiffest Rectangular Beam containing a given amount of materia that in which the ratio of depth to breadth is as 10 is to 6; hence, in desigr beams, the depth and breadth should be made to approach as near this rati is practicable.

Example 1. What is the greatest distributed load that an 8 by 10 in wh pine girder, 20 ft in span, will support, without deflecting at the center m than one-thirtieth of an inch per foot of span?

Solution. This girder comes under the case of a beam supported at b ends and loaded with a uniformly distributed load, and hence should be ca lated by Formula (10). Substituting the given dimensions in Formula (

$$\text{Safe load} = \frac{8 \times 8 \times 1\,000 \times 77}{5 \times 400} = 2\,464 \text{ lb}$$

Example 2. What should be the dimensions of a long-leaf yellow-pine be 10 ft in span, to support a concentrated load of 4 250 lb at the middle with deflecting more than one-third of an inch at the center?

Solution. A deflection of one-third of an inch in a span of 10 ft is in the p portion of one-thirtieth of an inch per foot of span; and as the load is con trated at the middle, Formula (7) should be used, with e_1, the value given in fourth column opposite long-leaf yellow pine.

Formula (7) gives the dimensions of the breadth, but in order to obtain a value for the depth must first be assumed. For such a short span, 10 inc would seem to be a proper depth.

Substituting in Formula (7)

$$\text{Breadth} = \frac{4\,250 \times 100}{1\,000 \times 116} = 3.6 \text{ in}$$

Hence it will be necessary to use a 4 by 10-in beam. As the span of this beam in feet is the same as its depth in inches, it should be tested to see if it meets the requirements for strength also. From Table XII, page 643, it is found that the safe distributed load for a 1 by 10-in beam, 10 ft in span, is 1 333 lb, and for a 4 by 10-in beam the safe load would be four times as much, or 5 332 lb. The load in this example, however, is applied at the middle; hence the safe distributed load must be divided by 2, which gives 2 666 lb for the safe load at the middle. As this is much less than the load to be carried, the size of the beam should be increased to 4 by 16 in. In general it is not safe to use the FORMULAS FOR STIFFNESS when the span in feet does not exceed the depth in inches.

Example 3. What is the largest load that an inclined spruce beam 8 by 12 in in cross-section and 16 ft in length between the supports will carry at the middle, consistent with stiffness, the horizontal distance between the supports being 14 ft?

Solution. Formula (12) is the one to be used in this case. Assuming the limit of deflection at one-thirtieth of an inch per foot of span, the value of e_1 is found opposite spruce in the last column of Table I. Making the proper substitutions,

$$\text{Safe load} = \frac{8 \times 1\,728 \times 92}{16 \times 14} = 5\,678 \text{ lb}$$

4. Cylindrical Beams

Formulas. The formulas for beams with SQUARE CROSS-SECTIONS may be used for beams with CIRCULAR CROSS-SECTIONS, if $1.7 \times e$ is substituted for e. That is, other conditions being equal, a CYLINDRICAL BEAM bends or deflects 1.7 times as much as a beam the cross-section of which is the square circumscribing the circular cross-section of the cylindrical beam.

5. Safe Loads for Wooden Beams for a Given Deflection

Use of Tables and Formulas. Tables VII to XV, inclusive, pages 638 to 646, giving the SAFE LOADS FOR BEAMS, give, also, the maximum loads for beams, 1 in thick, that will cause a DEFLECTION not exceeding 1/360 of the span, that is, 1/30 in per foot of span. Where two loads are given for any span or depth the upper load is calculated for STRENGTH and the lower load for DEFLECTION. Where one load is given the calculation is for strength, as the calculation for deflection in those particular beams would give an excessive load (Example 2). To find the corresponding load for any thickness, multiply the load given in the table by the breadth of the beam in inches. Suppose, for example, that it is required to find the greatest distributed load that an 8 by 10-in white-pine girder, 20 ft in span, will support, without deflecting at the center more than one-thirtieth of an inch per foot of span. Referring to Table VIII, page 639, giving the safe loads in pounds for white-pine beams, two values are found opposite the 20-ft span, 389 and 308, the latter being the safe load for deflection. The safe load, therefore, for an 8 by 10-in girder will be eight times, or $308 \times 8 = 2\,464$ lb, which agrees with the safe load for the same girder calculated for deflection by Formula 10, Example 1.

6. Nominal and Standard Sizes of Wooden Beams

Conversion Factors for Wooden Beams of Standard Size. Table II may be used for beams that measure less than the NOMINAL DIMENSIONS. DRESSED BEAMS, and in many localities floor-joists carried in stock, are more

or less SCANT of the nominal dimensions, and for such joists a reduction in t safe load must be made to correspond to the reduction in size. The DRESS SIZES are generally ¼ in scant up to 4 in in breadth, above which they are ½ scant; while in depth they are all generally ½ in less than the NOMINAL SIZ The safe loads may be obtained by multiplying the safe loads as given Tables VII to XV, pages 638 to 646, by the FACTORS given in the followi table:

Table II. Conversion Factors for Beams of Commercial or Standard Sizes

Cross-sections of beams in inches	Factors	Cross-sections of beams in inches	Factors
1¾×5½	1.47	1¾×11½	1.61
2¾×5½	2.31	2¾×11½	2 53
1¾×6½	1.51	1¾×13½	1 63
2¾×6½	2.51	2¾×13½	2.56
1¾×7½	1.54	1¾×15½	1.65
2¾×7½	2.42	2¾×15½	2.58
1¾×9½	1.58	1¾×17½	1.65
2¾×9½	2.48	2¾×17½	2.60

Example 4. What is the safe load for a 2¾ by 13½-in spruce beam, 18 span?

Solution. From Table VIII, page 639, the safe load for a 1 by 14-in be is 847 lb. Multiplying this by 2.56 the product is 2 168 lb, the safe distribut load for a beam 2¾ by 13½ in in cross-section. For a full, "nominal" si 3 by 14-in, the safe load would be 2 541 lb.

7. Deflection of Steel Beams

General Formula. The DEFLECTION of any steel beam may be found means of Formula (1), page 663.

Example 5. It is required to determine the deflection of a 12-in 31.8 beam, 20 ft in span, under its maximum uniformly distributed load 9.59 tons.

Solution. The load in pounds = 9.59 tons × 2 000 = 19 180 lb; the span inches = 20 ft × 12 = 240 in; c, for a beam supported at both ends and u formly loaded, from the values given under Formula (1), is 0.013; E, for stee 29 000 000 lb per sq in (Table I, page 664); and the moment of inertia, fr the properties of steel I beams, page 355, is 215.8. Substituting these val in Formula (1), page 663,

$$\text{Deflection in inches} = \frac{19\,180 \times 240^3 \times 0.013}{29\,000\,000 \times 215.8} = 0.551 \text{ in}$$

The allowable deflection is $\frac{1}{30}$ of an inch per foot of span, or $\frac{20}{30} = 0.666$ in.

Coefficients of Deflection. In order to save the time required to use t DEFLECTION-FORMULA, COEFFICIENTS OF DEFLECTION have been worked out f different spans and are given in Table III.

Table III. Coefficients of Deflection for Uniformly Distributed Loads*

Span in feet	Fiber-stress, pounds per square inch			Span in feet	Fiber-stress, pounds per square inch		
	16 000	14 000	12 500		16 000	14 000	12 500
1	0.017	0.014	0.013	26	11.189	9.790	8.741
2	0.066	0.058	0.052	27	12.066	10.558	9.427
3	0.149	0.130	0.116	28	12.977	11.354	10.138
4	0.265	0.232	0.207	29	13.920	12.180	10.875
5	0.414	0.362	0.323	30	14.897	13.034	11.638
6	0.596	0.521	0.466	31	15.906	13.918	12.427
7	0.811	0.710	0.634	32	16.949	14.830	13.241
8	1.059	0.927	0.828	33	18.025	15.772	14.082
9	1.341	1.173	1.047	34	19.134	16.742	14.948
10	1.655	1.448	1.293	35	20.276	17.741	15.841
11	2.003	1.752	1.565	36	21.451	18.770	16.759
12	2.383	2.086	1.862	37	22.659	19.827	17.703
13	2.797	2.448	2.185	38	23.901	20.913	18.672
14	3.244	2.839	2.534	39	25.175	22.028	19.668
15	3.724	3.259	2.909	40	26.483	23.172	20.690
16	4.237	3.708	3.310	41	27.823	24.346	21.737
17	4.783	4.186	3.737	42	29.197	25.548	22.810
18	5.363	4.692	4.190	43	30.604	26.779	23.909
19	5.975	5.228	4.668	44	31.954	28.039	25.034
20	6.621	5.793	5.172	45	33.517	29.328	26.185
21	7.299	6.387	5.703	46	35.023	30.646	27.362
22	8.011	7.010	6.259	47	36.562	31.992	28.565
23	8.756	7.661	6.841	48	38.135	33.368	29.793
24	9.534	8.342	7.448	49	39.741	34.773	31.047
25	10.345	9.052	8.082	50	41.379	36.207	32.328

*Taken by permission from Pocket Companion, Carnegie Steel Company.

To find the deflection in inches of a section SYMMETRICAL ABOUT THE NEUTRAL AXIS, such as the section of an I beam, channel, zee, etc., divide the coefficient in the table corresponding to the given span and fiber-stress by the depth of the section in inches. To find the deflection in inches of a section NOT SYMMETRICAL ABOUT THE NEUTRAL AXIS, such as the section of an angle, tee, etc., divide the coefficient corresponding to the given span and fiber-stress by twice the distance of the extreme fiber from the neutral axis, obtained from the tables of Chapter X. To find the deflection in inches of a section FOR ANY OTHER FIBER-STRESS than the fiber-stresses given, multiply this fiber-stress by any of the coefficients in Table III, for the given span, and divide by the fiber-stress corresponding to the coefficient used.

Example 6. Required the deflection of a 10-in 25-lb beam of 10-ft span, under its maximum distributed load of 13 tons, the fiber-stress being taken at 16 000 lb per sq in. Table III gives 1.655 as the deflection-coefficient, and dividing this by 10, the depth of the beam in inches, the result is $1.655/10 = 0.1655$, for the deflection at the middle. By Formula (1), page 663, the deflection for the same beam, span, and load $= \dfrac{26\,000 \times 1\,728\,000 \times 0.013}{29\,000\,000 \times 122.1} = 0.1649$ in, the

two results being nearly identical. For the same beam, a span of 18 ft and load of 7.2 tons, the deflection by the table is 0.5363 in; and by Formula (1 0.5328 in, practically the same result.

Safe Loads and Deflection. In the tables of Chapter XV, giving the sa loads for I beams, channels and rolled beams of other cross-sections, the loa given are for the SAFE LIMIT OF DEFLECTION; and the safe loads, also, are giv which will cause deflections of more than 1/360 of the span-length in inches.

Lateral Deflection of Beams. When the unbraced length exceeds t times the width, the tabular safe loads should be reduced in accordance wi the ratios given in the following table in order to insure that the stresses in t compression-flanges should not exceed the allowed safe unit stress:

Length of span	Allowable safe load	Length of span	Allowable safe loa
5 × flange-width	Full tabular load	25 × flange-width	71.9% tabular loa
10 × flange-width	Full tabular load	30 × flange-width	62.5% tabular loa
15 × flange-width	90.6% tabular load	35 × flange-width	53.1% tabular loa
20 × flange-width	81.2% tabular load	40 × flange-width	43.8% tabular loa

"In addition to this lateral deflection which is induced within the beam the action of pure bending-stresses, lateral deflection may be induced by t thrust of floor-arches or other loading acting on an axis perpendicular to t line of principal bending-stress. The thrust of these arches should either neutralized by tie-rods, or the safe carrying capacity of the beam should computed in accordance with the general formulas of flexure to provide the combined stresses due to the action of both vertical and horizontal forc that is to say, the safe loads should be figured around both the axes 1-1 a 2-2, and the unit stress computed so as not to exceed 16 000 lb per sq in."

8. Graphical Determination of Deflection of Beams

The Deflection of a Beam with parallel flanges and constant moment inertia may be DETERMINED GRAPHICALLY. The deflected form is identi with the bending-moment curve for t beam with a load distributed in a fo similar to that of the bending-mome diagram. Fig. 1 is a beam of length L. The moment-curve due to loadi ABC is the deflection-curve due concentrated load P. The deflecti curve is obtained graphically by div ing area ABC into thin vertical str and constructing force and equilibriu polygons (page 296). If a pole-distan be chosen bearing a convenient ra to EI (E is the Modulus of Elastic of the material and I the Moment Inertia of the cross-section of the bea the deflection at any point of the be will have the same ratio to the sca ordinate at that point of the equilibrium-polygon.

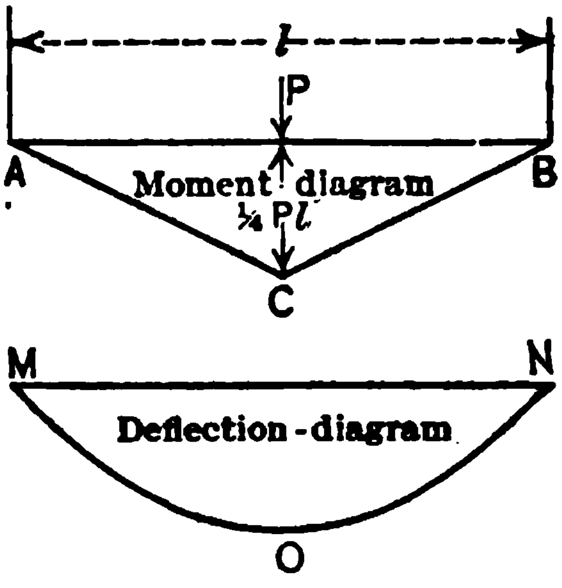

Fig. 1. Moment and Deflection-diagrams of Beam Loaded at the Middle

CHAPTER XIX

STRENGTH AND STIFFNESS OF CONTINUOUS GIRDERS

BY

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1. General Considerations

ontinuous Versus Single-Span Girders. A CONTINUOUS GIRDER is one ing upon three or more supports, as distinguished from a SIMPLE GIRDER ich rests upon two supports. Continuous girders, except in reinforced-con- te construction, and in some types of grillage-foundations, are of rare occurre in building-construction. While in almost every building of importance necessary to employ girders resting upon piers or columns, placed from 15 20 ft apart, and while in many cases steel girders could conveniently be obed which would span two and even three of the bays between the supports, y are practically limited to one-story buildings, because in tall buildings it is ter construction to have the vertical rather than the horizontal supports tinuous. Many different opinions are held as to the RELATIVE STRENGTH STIFFNESS of continuous and non-continuous girders, and different formulas e been proposed from time to time; but in this chapter the mathematical ussions will not be given.*

Continuous Girders and Overhanging Girders. In all CONTINUOUS DERS, the end-spans (Fig. 2) are somewhat in the condition of a SIMPLE DER with ONE OVERHANGING END, while the other spans are somewhat in condition of a SIMPLE GIRDER with TWO OVERHANGING ENDS. At each interate support there is a NEGATIVE BENDING MOMENT, the effect of which reduce the bending moments between the supports.

2. Supporting Forces or Reactions of Continuous Girders

ntinuous Girder of Two Equal Spans. Concentrated Load at the dle of Each Span. If a girder of two spans, each equal to l (Fig. 1), be ed at the middle of the left span with P lb, and at the middle of the right with P_1 lb, the reaction at the support R_1 is determined by the formula

$$R_1 = \frac{13\,P - 3\,P_1}{32} \qquad (1)$$

reaction at the support R_2 by

$$R_2 = {}^{11}\!/_{16}\,(P + P_1) \qquad (2)$$

the reaction at the support R_3 by the formula

$$R_3 = \frac{13\,P_1 - 3\,P}{32} \qquad (3)$$

* For the derivation of the following formulas, see an article by F. E. Kidder on this ect, in Van Nostrand's Engineering Magazine, July, 1881.

If $P = P_1$, then each of the end-supports must support $\frac{5}{16} P$ and the mi… support $\frac{22}{16} P$. If the girder is cut so as to make two girders of one … each, then the end-supports will carry $\frac{1}{2} P$ or $\frac{8}{16} P$, and the middle sup… $\frac{16}{16} P$. Hence, it is seen that by … the continuous girder of three spans … reactions of the end-supports are di… ished, while the reaction at the mi… support is increased.

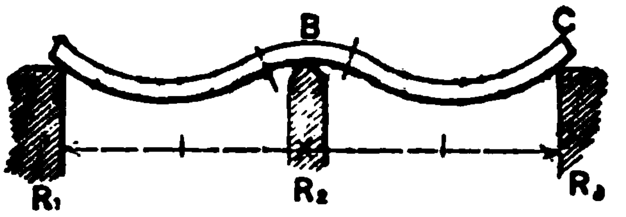

Fig. 1. Continuous Girder of Two Spans

Continuous Girder of Two Sp… Uniformly Distributed Load Over … Span (Fig. 1). Load over each span equals w lb per unit of length. … be the length of the left span and l_1 the length of the right span. Reactio… left support

$$R_1 = \frac{w}{2}\left[l - \frac{l_1^3 + l^3}{4l\,(l + l_1)}\right]$$

Reaction of middle support,

$$R_2 = w\,(l + l_1) - R_1 - R_3$$

Reaction of right support

$$R_3 = \frac{w}{2}\left[l_1 - \frac{l_1^3 + l^3}{4l_1\,(l + l_1)}\right]$$

When both spans are equal to l, the reaction of each end-support is $\frac{3}{8}\,wl$, … of the middle support $\frac{5}{4}\,wl$; hence the girder, by being continuous, reduce… reactions of the end-supports, and increases that of the middle support … 25%.

Continuous Girder of Three Equal Spans. Concentrated Load of P Poun… the Middle of Each Span (Fig. 2).

Reaction of either abutment

$$R_1 = R_4 = \tfrac{7}{20}\,P$$

Reaction of either middle support

$$R_2 = R_3 = \tfrac{23}{20}\,P$$

or the reactions of the two end-supports are $\frac{3}{20}$ less, and those of… two middle supports $\frac{3}{20}$ greater than they would have been had three separate girders of the same cross-section been used, instead of one continuous girder.

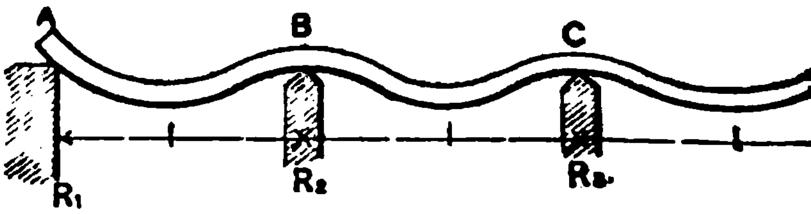

Fig. 2. Continuous Girder of Three Spans

Continuous Girder of Three Equal Spans. Uniformly Distributed Load… Each Span (Fig. 2). The load per unit of length is w lb.

Reaction of either end-support

$$R_1 = R_4 = \tfrac{2}{5}\,wl$$

Reaction of either middle support

$$R_2 = R_3 = \tfrac{11}{10}\,wl$$

Hence the reactions of the end-supports are $\frac{1}{5}$ less, and of the middle su… $\frac{1}{10}$ more, than if the girder were not continuous.

3. Bending Moments of Continuous Girders

Strength of Continuous Girders. The STRENGTH of a girder depends upon its aterial and the shape of its cross-section, and also upon the disposition of the ernal loads imposed upon it. The latter give rise to the BENDING MOMENTS, ich are measures of the tendencies of the external forces, such as the loads d the supporting forces, to bend or to break the girder. It is the difference the numerical values of these BENDING MOMENTS which causes the difference the FLEXURAL STRENGTH of continuous and non-continuous girders of the e cross-section.

Continuous Girders of Two Spans. When a beam is at the point of breaking y flexure, the FLEXURE-FORMULA, $M = SI/c$, is frequently used to calculate a MISAL UNIT STRESS developed in the beam; and when the beam has a rec-gular cross-section the formula takes the form (see page 635)

$$\text{Maximum bending moment} = \frac{\text{Modulus of rupture}^* \times \text{breadth} \times \text{square of depth}}{6} \qquad (11)$$

order that the beam may carry its load with perfect safety, the breaking-load st be divided by a proper FACTOR OF SAFETY. Hence, if the MAXIMUM DING MOMENT of a beam can be found under any conditions, the required ensions of the beam can easily be determined from Formula (11). (See le I, page 557, for the safe values of the FIBER-STRESSES.) The greatest ding moment for a continuous girder of two spans is almost always over the dle support, and is a MINUS BENDING MOMENT, if the plus sign is given to maximum bending moments between the supports. It is the NUMERICAL UE only, however, that is considered.

Continuous Girder of Two Spans. Distributed Load over Each Span. The atest bending moment in a continuous girder of two spans, l and l_1 (Fig. 1), ded with a uniformly distributed load of w lb per unit of length is over the ddle support and is

$$\text{Maximum bending moment} = \frac{wl^3 + wl_1^3}{8\,(l + l_1)} \qquad (12)$$

en $l = l_1$, or both spans are equal,

$$\text{Maximum bending moment} = wl^2/8 \qquad (12a)$$

ch is the same as the maximum bending moment of a beam supported at ends and uniformly loaded over its whole length. Hence a continuous r of two spans uniformly loaded is no stronger as far as flexure is concerned if non-continuous.

ontinuous Girder of Two Equal Spans. Concentrated Load at the Middle Each Span. The greatest bending moment in a continuous girder of two l spans, each of length l, loaded with P lb at the middle of one span, and P_1 lb at the middle of the other, is

$$\text{Maximum bending moment} = \tfrac{5}{32}\, l\,(P + P_1) \qquad (13)$$

* The modulus of rupture is equal to the ultimate flexural unit stress developed in a m when the bending moment is great enough to cause failure, and is expressed in nds per square inch. It usually lies between the ultimate unit compressive strength the ultimate unit tensile strength of the material. (See, also, Chapter XV, page) It is to be noted, that the flexure-formula $M = SI/c$ is not really applicable to s of materials for which the stresses are not proportional to the deformation, nor to homogeneous beams, nor to beams under stresses greater than the elastic limit of material.

When $P = P_1$, or the two loads are equal, this becomes

$$\text{Maximum bending moment} = \tfrac{3}{16}\,Pl \qquad (1$$

or ¼ less than its value when the beam is cut at the middle support.*

Continuous Girder of Three Spans. Uniformly Distributed Load Over E Span. The greatest bending moment in a continuous girder of three sp loaded with a uniformly distributed load of w lb per unit of length, the len of each end-span being l_1 and of the middle span l, is at either of the middle s ports, and is determined by the formula

$$\text{Maximum bending moment} = \frac{wl^3 + wl_1^3}{4\,(3\,l + 2\,l_1)}$$

When the three spans are equal, this becomes

$$\text{Maximum bending moment} = wl^2/10 \qquad ($$

or ⅕ less than what it would be were the beam not continuous.

Continuous Girder of Three Equal Spans. Concentrated Load of P Poun the Middle of Each Span. The greatest bending moment in a contin girder of three equal spans, each of a length l, and each loaded at the mi with P pounds, is

$$\text{Maximum bending moment} = \tfrac{3}{20}\,Pl$$

or ⅖ less than that of a non-continuous girder.

4. Deflection of Continuous Girders

Continuous Girder of Two Equal Spans. Uniformly Distributed Load Each Span. The greatest deflection of a continuous girder of two equal s loaded with a uniformly distributed load of w lb per unit of length is

$$\text{Maximum deflection} = 0.005416\,wl^4/EI$$

in which E is the MODULUS OF ELASTICITY and I the MOMENT OF INERTIA c cross-section of the beam. The greatest deflection of a similar beam supp at both ends and uniformly loaded is

$$\text{Maximum deflection} = 0.013020\,wl^4/EI$$

Hence the deflection of the continuous girder is only about ⅖ that of a continuous girder. The greatest deflection of a continuous girder of two is not at the middle of either span, but between the middle point of a spa one of the abutments. The greatest deflection of a continuous girder c equal spans, loaded at the middle of one span with a load of P lb, and a middle of the other with P_1 lb, is, for the span with the load, P

$$\text{Maximum deflection} = \frac{(23\,P - 9\,P_1)\,l^3}{1\,536\,EI}$$

for the span with load P_1

$$\text{Maximum deflection} = \frac{(23\,P_1 - 9\,P)\,l^3}{1\,536\,EI}$$

When both spans have the same load

$$\text{Maximum deflection} = \tfrac{7}{768}\,Pl^3/EI$$

* In this continuous beam the maximum bending moment is the minus bendi ment over the middle support and in each of the two simple beams the maximum ing moment is a plus bending moment and is between two supports.

The greatest deflection of a simple beam supported at both ends and loaded at the middle with P lb is

$$\text{Maximum deflection} = Pl^3/48\,EI$$

so the deflection of the continuous girder is only 7/16 that of a non-continuous one.

Continuous Girder of Three Equal Spans. Uniformly Distributed Load Over Each Span.
The load per unit of length is w lb.

$$\text{Greatest deflection at the middle of middle span} = 0.00052\,wl^4/EI \qquad (18)$$
$$\text{Greatest deflection in the end-spans} = 0.006884\,wl^4/EI \qquad (19)$$

Hence the maximum deflection of the continuous girder is only about ½ that of a non-continuous girder.

Continuous Girder of Three Equal Spans. Concentrated Load P at the Middle of Each Span.

$$\text{Greatest deflection at the middle span} = 1/480\; Pl^3/EI \qquad (20)$$
$$\text{Greatest deflection at the middle of end-spans} = 11/960\; Pl^3/EI \qquad (21)$$

Hence the maximum deflection of the continuous girder is only 11/20 of that of a non-continuous girder.

Notes on Reactions, Strength and Stiffness of Continuous Girders

Supports and Reactions of Continuous and Non-Continuous Girders. From the foregoing, some conclusions can be drawn which will be of use in deciding whether it is best in any case to use a CONTINUOUS or a NON-CONTINUOUS GIRDER. From the formulas given for the reactions of the supporting forces of the different cases of continuous girders it is seen that the end-supports do not bear as much of the load as they do when the girders are non-continuous. The difference is added to the reactions of the other supports. This might be an advantage in a building in which the girders run across the building, and have their outside ends supported by the side walls and their inside ends by piers or columns. In this case, by using continuous girders, part of the load could be taken from the walls and transferred to the piers or columns. But in cases of this kind, the vibration may have to be considered. If the building is a mill or factory in which the girders support machines, any vibration in the middle span of the girder is carried to the side walls if the girder is continuous; while if non-continuous girders are used, with their ends an inch or so apart, little or no vibration is carried to the side walls from the middle span. In all cases of important construction the supporting forces should be carefully considered.

Relative Strength of Continuous and Non-Continuous Girders. As the RELATIVE STRENGTH of continuous and non-continuous girders of the same cross-section, material and spans, and loaded in the same way, is proportional to their maximum BENDING MOMENTS, the strength of a continuous girder can be calculated, from the formula for its MAXIMUM BENDING MOMENT. From the formulas given for these bending moments for the various cases considered, it is seen that the parts of the girder most stressed are those which come over the middle supports. It is seen, also, that, except in the single case of a girder of two spans uniformly loaded, the strength of a continuous girder is greater than that of a non-continuous girder. But the gain in strength in some instances is not very great, although it is generally enough to pay for making the girder continuous.

Relative Stiffness of Continuous and Non-Continuous Girders. The STI NESS of a girder varies inversely as its DEFLECTION; that is, the less the def tion under a given load the stiffer the girder. From the values given for MAXIMUM DEFLECTION of continuous girders, it is evident that the STIFFNESS a girder is increased by making it continuous; and this is usually the princi advantage in the use of continuous girders. It sometimes happens in buildi construction that it is necessary to use beams and girders of much grea strength than is required to carry the superimposed load, because the deflecti of smaller beams or girders would be too great. But if continuous gir are used they may be made of just the size required for strength, because t deflections are less. Where great stiffness is required, therefore, continu beams or girders should be used if possible, as in the case of grillage-gird (See Example 3, page 679.)

6. Formulas for the Strength and Stiffness of Continuous Girde

Girders of Rectangular Cross-Section. For convenience, the proper form for calculating the strength and stiffness of continuous girders of rectang cross-section are given. The formulas for strength are deduced from the flex formula $M = SI/c$, modified for the rectangular section of breadth b and dept

$$\text{Bending moment} = \frac{b \times d^2 \times S}{6}$$

in which S is the safe unit fiber-stress. This is eighteen times the coeffic A * of Table II, page 628.

STRENGTH. **Continuous Girder of Two Equal Spans. Uniformly Distrib Load Over Each Span.**

$$\text{Breaking-load} \dagger = \frac{2 \times b \times d^2 \times A^*}{l}$$

where b denotes the breadth and d the depth of the girder in inches, and l length of one span, in feet. The values of the constant A are three times values given in Table II, page 628. For long-leaf yellow pine, 201; for Dou fir, 168; chestnut, 132; and for spruce and white pine, 117 lb per sq in, are ommended for the values of A in these formulas.

Continuous Girder of Two Equal Spans. Concentrated Load at the Midd Each Span.

$$\text{Breaking-load} = \tfrac{4}{3} \times \frac{b \times d^2 \times A}{l}$$

Continuous Girder of Three Equal Spans. Uniformly Distributed Load Each Span.

$$\text{Breaking-load} = \tfrac{5}{4} \times \frac{b \times d^2 \times A}{l}$$

Continuous Girder of Three Equal Spans. Concentrated Load at the Mi of Each Span.

$$\text{Breaking-load} = \tfrac{5}{3} \times \frac{b \times d^2 \times A}{l}$$

* See, also, Table I, page 557, and Table XVI, page 647, for safe fiber-stresses.
† Breaking-load in pounds in all cases.

FFNESS. Continuous Girder of Two Equal Spans. Uniformly Distributed d Over Each Span.

e following formulas give the loads which the beams will support without cting more than one-thirtieth of an inch per foot of span.

$$\text{Load on one span} = \frac{b \times d^3 \times e_1}{0.26 \times l^2} \tag{27}$$

ntinuous Girder of Two Equal Spans. Concentrated Load at the Middle of h Span.

$$\text{Load on one span} = {}^{16}\!/_{9} \times \frac{b \times d^3 \times e_1}{l^2} \tag{28}$$

Continuous Girder of Three Equal Spans. Uniformly Distributed Load Over h Span.

$$\text{Load on one span} = \frac{b \times d^3 \times e_1}{0.33 \times l^2} \tag{29}$$

ntinuous Girder of Three Equal Spans. Concentrated Load at the Middle ch Span.

$$\text{Load on one span} = {}^{20}\!/_{11} \times \frac{b \times d^3 \times e_1}{l^2} \tag{30}$$

he value of the constant e_1 is obtained by dividing the MODULUS OF ELAS- TY by 12 960; and, for the three woods most commonly used as beams, the wing values may be taken:

ong-leaf yellow pine, 116; white pine, 77; spruce, 92; Douglas fir, 116. other woods, see Table I, page 664.)

r Continuous Steel Beams the requisite size may be found by first com- ing the MAXIMUM BENDING MOMENT, by means of Formulas (12) to (15), then selecting a beam that has a

$$\text{SECTION-MODULUS} = \frac{3 \times \text{maximum bending moment in ft-lb}}{4\,000} \tag{31}$$

lues for the SECTION-MODULI for the different shapes of rolled steel used as s are given in the tables in Chapter X.

mple 1. What steel beam should be used to support two loads of 16 000 ch, concentrated at the middle of two spans of 10 ft each, the beam being nuous?

tion. Formula (13*a*) gives the maximum bending moment as $\frac{3}{16}$ *Pl*, or 0 ft-lb. Therefore, from Equation (31), a beam having a section-modulus l to 3 × 30 000/4 000 or 22.5 should be used. From the Table IV, page it is found that a 9-in 30-lb beam has a section-modulus of 22.5, and a 25.4-lb beam a section-modulus of 24.4. Either of these beams will there- answer, the 10-in beam being the cheaper, however, and also the stiffer.

mple 2. A steel beam continuous over three spans is required to support ormly distributed load of 1 000 lb per lin ft. The two end-spans are 12 ft , and the middle span 10 ft. What should be the size and the weight of the n used?

tion. The maximum bending moment is found by Formula (14), and is

$$\frac{1\,000 \times 1\,000 + 1\,000 \times 1\,728}{4\,(30 + 24)} = 12\,630$$

The section-modulus, by Equation (31), must equal $3 \times 12\,630/4\,000 =$ which requires a 7-in 15.3-lb beam (Table IV, page 355).

If the beam were not continuous an 8-in 18.4-lb beam would be require the 12-ft spans, and a 7-in 15.3-lb beam for the 10-ft span.

For a beam of two equal spans, loaded uniformly, the strength is the san though it were not continuous.

The formulas given for the reactions at the supports, and for the deflec of continuous girders with concentrated loads, were verified by Mr. Kidd means of careful experiments on small steel bars. The remaining forr were verified by comparing them with the formulas of other authorities it was possible to do so. In regard to some of the cases given the autho never seen any discussion of them in any work on the subject.

7. Continuous Girders in Grillage-Foundations

Grillage-Beams Considered as Inverted Continuous Girders. As stat the beginning of this chapter, CONTINUOUS GIRDERS, as such, are seldom us building-construction, although their employment in grillage-beam footings i

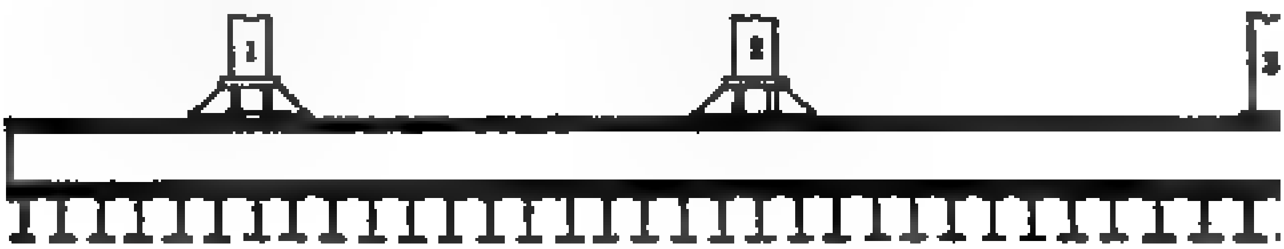

Fig. 3. Continuous Girder in Grillage-foundation

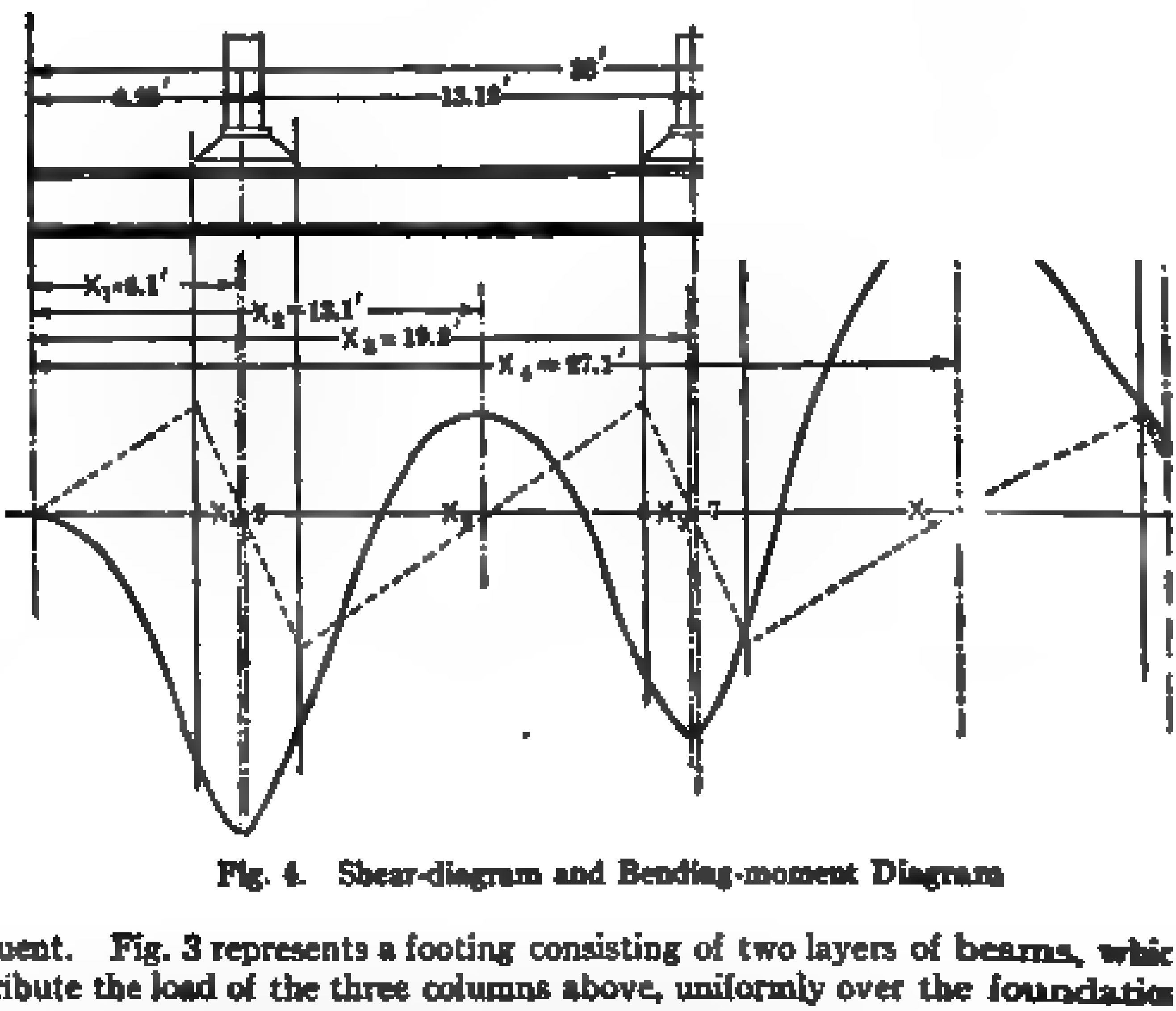

Fig. 4. Shear-diagram and Bending-moment Diagram

quent. Fig. 3 represents a footing consisting of two layers of beams, whic tribute the load of the three columns above, uniformly over the foundatio By inverting the footing the three columns become the supports or reaction the upper layer of beams, a continuous girder, loaded with a uniformly di uted load which is the pressure of the lower layer. As in practice the co

loads are never equal, and the distance between the columns seldom equal, it is necessary to project the continuous girder beyond the most heavily loaded column in order to insure a uniform pressure upon the lower layer. Because of these limitations none of the formulas previously deduced can be applied, although the principles upon which they are based hold good.

Maximum Bending Moment. Since the REACTIONS in this case are the given column-loads it is required first to find the MAXIMUM BENDING MOMENT. From what has already been said about continuous girders, it is evident that the point of maximum bending moment may be under columns 1 or 2, or between the columns. Since the maximum bending moments are the POINTS OF NO SHEAR, construct the SHEAR-DIAGRAM, find where the shear passes through zero, and calculate the bending moments at these points. The maximum bending moment is determined, as in examples 1 and 2, in order to determine the SECTION-MODULUS of the girder.

Example 3. The continuous girder under columns 1, 2 and 3 (Fig. 3) is 33 ft long; the overhang, to the left of column 1, 6.25 ft; the distance between columns 1 and 2, 13.12 ft; between columns 2 and 3, 12.88 ft; and from column 3 to the right edge of the girder, .75 ft. The column-loads are as follows: on column 1, 565 tons; on column 2, 600 tons; and on column 3, 255 tons.

The column-loads may be considered uniformly distributed over parts of the girder by the bases, which are 3 ft wide under columns 1 and 2 and 18 in wide under column 3. The UNIT PRESSURE under column 1, therefore, is $565/3 = 188.3$ tons; under column 2, $600/3 = 200$ tons; and under column 3, $255/1.5 = 170$ tons. The unit pressure under the continuous girder is

$$(565 + 600 + 255)/33 = 43 \text{ tons}$$

The first step in the calculation of the girder is the determination of the POINTS OF NO SHEAR and the plotting of the SHEAR-DIAGRAM in Fig. 4. It is obvious from the shear-diagram that there are four points of no shear and consequently four points of POSSIBLE MAXIMUM BENDING MOMENT. The first of these is under column 1, the second between columns 1 and 2, the third under column 3 and the fourth between columns 2 and 3. The bending-moment diagram is shown by the solid curved line in Fig. 4. The points of contraflexure or no bending moment are the intersections of this line with the horizontal line of reference.

The SHEAR-DIAGRAM,* shown by the broken line in Fig. 4, may be constructed as follows:

$$V_1\dagger = +43 \text{ tons per ft} \times (6.25 - 1.5 = 4.75 \text{ ft}) = +204.25 \text{ tons}$$

$$V_2 = (+43 \text{ tons per ft} \times 6.25 \text{ ft}) - 565/2 \text{ tons} = +268.75 - 282.5 = -13.75 \text{ tons}$$

This shows that x_1, the point of no shear, lies between points 1 and 2. To find this point let y be its distance beyond or to the right of point 1. Then, the EQUATION FOR NO SHEAR is 43 tons $\times$ (4.75 ft + y ft) = $188.3 \times y$, or $204.25 + 43\,y = 188.3\,y$, from which $145.3\,y = 204.25$ and $y = 1.4$ ft: hence x_1, the FIRST POINT OF NO SHEAR, is 4.75 ft + 1.4 ft, or 6.1 ft from the left end.‡

The SECOND POINT OF NO SHEAR, x_2, is such a distance from the left end that the DOWNWARD SHEARING-FORCE of 565 tons from column 1 is neutralized by

* The upward forces are here called plus or positive and the downward forces minus or negative.

† V_1 is taken at point 1, the left edge of base of Column 1, V_2 at point 2, at the axis of column 2, etc.

‡ The following computations are carried out to one decimal-place, only, the nearest approximate values being used.

an equal UPWARD SHEARING-FORCE of 43 tons per ft. on x_3 ft. Hence $x_2 = 565/$
$= 13.1$ ft.

$V_6 = +43$ tons per ft $\times$ [(6.25 + 13.12 − 1.5) = 17 9 ft] − 565 tons
$= 769.7 - 565 = +204.7$ tons

$V_7 = +43$ tons per ft $\times$ (6.25 + 13.12 = 19.4 ft) − (565 + 600/2 tons)
$= +834.2 - 865$ tons $= -30.8$ tons

This shows that the THIRD POINT OF NO SHEAR, x_3, lies between 6 and 7. L
y be its distance to the right of point 6. The equation for no shear at th
point is 43 tons $\times$ (17.9 ft + y ft) = 565 tons + (200 tons $\times$ y ft), or 769.7 + 43
= 565 + 200 y. from which 157 y = 204.7 and y = 1.3 ft. Hence x_3, the THI
POINT OF NO SHEAR, is 17.9 ft + 1.3 ft = 19.2 ft from the left end.

The FOURTH POINT OF NO SHEAR, x_4, is such distance from the left end th
the DOWNWARD SHEARING-FORCE of columns 1 and 2, amounting to 565 + 6
or 1165 tons, is neutralized by an equal UPWARD SHEARING-FORCE of 43 to
per ft on x_4 ft. Hence $x_4 = 1\,165/43 = 27.1$ ft.

Having found the points of no shear, the BENDING MOMENT at these poin
may now be determined.

M at $x_1 = 43$ tons $\times$ 6.1 ft $\times$ 6.1/2 ft − 188.3 tons $\times$ 1.4 ft $\times$ 1.4/2 ft
$= +615.5$ ft-tons

M at $x_2 = 43$ tons $\times$ 13.1 ft $\times$ 13.1/2 ft − 565 tons $\times$ 6.8 ft $= -152.4$ ft-to

M at $x_3 = 43$ tons $\times$ 19.2 ft $\times$ 19.2/2 ft − 565 tons $\times$ 12.9 ft − 200 T $\times$ 1.3
$\times$ 1.3/2 = +467 ft-tons

M at $x_4 = 43$ tons $\times$ 27.1 ft $\times$ 27.1/2 ft − 565 tons $\times$ 20.8 ft − 600 tons
$\times$ 7.7 ft $= -582.2$ ft-tons

The MAXIMUM BENDING MOMENT therefore is at x_1 and equals 615.5 ft-tons*
1 231 000 ft-lb. Substituting in Formula (31), the SECTION-MODULUS is foun
to be $\frac{3 \times 1\,231\,000}{4\,000} = 923.2$. The following beams could be used, as far
flexure is concerned. For investigations of the resistance of the girders to we
buckling or crippling, see Chapter II, pages 182 to 184, and Chapter X
pages 567 to 569.

Four standard 24-in 110-lb I beams, section-modulus of each, 239.1 (page 35

Three Bethlehem 30-in 120-lb I beams, section-modulus of each, 349.3 (pa
357)

Three Bethlehem 24-in 140-lb girder-beams, section-modulus of each, 350
(page 358)

Two Bethlehem 28-in 180-lb girder-beams, section-modulus of each, 518
(page 358)

The 28-in and 30-in beams are stiffer than the 24-in beams, have a small
total amount of steel and cost less than the others for the number of bea
required.

* The bending moments at x_1 and x_4 have very nearly the same numerical values, a
in the computations the retaining or dropping of figures in the second decimal-place m
change the result and make the value at x_4 slightly greater than at x_1.

CHAPTER XX

RIVETED STEEL PLATE AND BOX GIRDERS

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1. General Notes on Plate and Box Girders

Types of Riveted Girders. Girders built up of plates and angles, as shown in section in Figs. 1 to 4, are extensively used. This is undoubtedly owing to the simplicity of their construction, to the comparatively low cost of the shapes of which they are fabricated and to their adaptability to any arrangement of loads or to any span for which girders are usually required. Riveted girders, however, are seldom made for spans greater than 60 ft and are seldom more than 6 ft in depth. The most common forms of these girders are shown in Figs. 2 and 4.

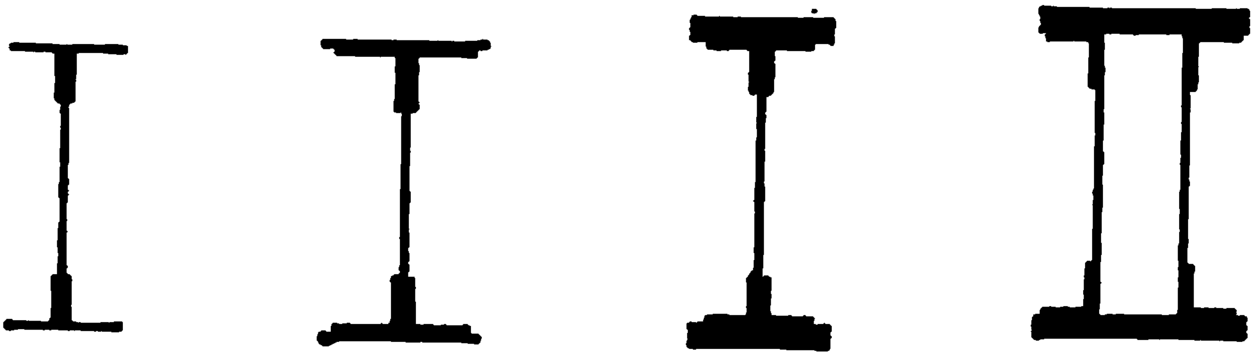


Fig. 1 Fig. 2 Fig. 3 Fig. 4

Types of Riveted Girders

The girders with a single vertical plate called the WEB are usually called PLATE GIRDERS, and those with double or triple webs, BOX GIRDERS. Plate girders are more economical than box girders, and more accessible for painting and inspection; but box girders are stiffer laterally and should always be used where great length of span requires wide flanges. In general, it may be said that plate girders should be used to support floor-beams and floor-arches and walls not over 12 in thick, and that box girders should be used where a flange-width greater than 12 in is required. The girder shown in section in Fig. 1 has no flange-plates and should be used only for comparatively light loads and short spans, and never to support masonry.

Flange and Web. The term FLANGE, as applied to riveted girders, includes all the metal in the top or bottom parts of the girder, exclusive of the web-plates.* The DEPTH of a riveted girder is the distance between the centers of gravity of the flanges; but in practice this is usually taken as the DEPTH OF THE WEB-PLATE, and the word will be so used in this chapter. The top and bottom of the flange-angles extend ¼ in beyond the top and bottom of the web-plate. (See the figure in Table IV, page 706.) STIFFENERS are short pieces of angles

* This may be modified, however, as some engineers include one-sixth of the web-area in the effective flange-area. See, also, Flange-Area in the examples of this chapter.

riveted to the web at intervals, to keep it from BUCKLING. They should f closely against the horizontal legs of the flange-angles, and should alwa be used at the supports and under concentrated loads.

Economic Depths of Girders. The depth of a riveted girder may va from one-tenth to one-fifteenth or, in exceptional cases, one-sixteenth the spa The greatest economy of material is said to be obtained when the depth is on twelfth the span. Thus for a 36-ft span a 3-ft girder should be used if tl conditions will permit; but the least depth should be 1/15 of 36, or about 2 5 in or, in exceptional cases, 1/16 of 36, or 2 ft 3 in. A girder is said to have i ECONOMIC DEPTH when the amount of material in the flanges is equal to th in the web, and there are no cover-plates. The rule holds approximate when there are cover-plates.

The Width of the Top Flange should not be less than one-twentieth tl distance between lateral supports; or if there are no lateral supports, then n less than one-twentieth the span.

Arches Between Girders, or floor-beams riveted to the sides of girders, m be considered as LATERAL SUPPORTS.

2. Details of Construction of Plate and Box Girders*

General Requirements for Plate and Box Girders. The following r quirements are those which must be generally satisfied in the design of rivet girders.

(1) All the connections and details of the several parts shall be of such streng that, upon testing, rupture shall occur in the body of the members rather th in any of their details or connections.

(2) In members subject to tensile stress full allowance shall be made for t reduction of the section by the rivet-holes.

(3) The webs of plate girders, when they cannot be obtained in one lengt must be spliced at all joints by a plate on each side of the web.

(4) Tees must not be used for splices.

(5) Stiffeners shall be used at the ends of all girders, wherever there are co centrated loads, and elsewhere when the shearing-stress is greater than t resistance to buckling.

(6). The PITCH, that is, the distance between centers of rivets, shall not e ceed 6 in, nor 16 times the thickness of the thinnest outside plate, and it sh not be less than 2¼ in for ¾-in rivets, or 2⅝ in for ⅞-in rivets, in a straig line.

(7) The rivets used should be ¾ in in diameter for plates from ⅜ to ⅝ thick, and ⅞ in in diameter for plates of greater thickness.

(8) The distance between the edge of any piece and the center of a riv hole must never be less than 1¼ in.

(9) In PUNCHING plates or other members, the diameter of the die shall in case exceed the diameter of the punch by more than 1/16 in.

(10) All RIVET-HOLES must be so accurately punched that when the seve parts forming one member are assembled, a rivet, 1/16 in less in diameter th the hole, can be inserted, hot, into any hole without REAMING or stressing t metal by the use of drift-pins.

(11) The rivets when driven must completely fill the holes.

(12) The RIVET-HEADS must be hemispherical, except where flush surfa are required, and of uniform size throughout for rivets of the same size. Th must be full and neatly made, and be concentric with the rivet-holes.

* These requirements are taken largely from Birkmire's "Compound Riveted Girder

(13) Whenever possible, all rivets must be MACHINE-DRIVEN.
(14) The several pieces forming one built member must fit closely together, d, when riveted, must be free from twists, bends or open joints.
(15) Girders 60 ft and less in length seldom require SPLICING, as the plates and gles can readily be obtained in such lengths. In splicing the top flange, n of two or more thicknesses, no additional COVER-PLATE will be required r the joint, but the ends should be planed true and butt closely. The rivets uld be spaced closer near the joint.
(16) The plate covering the bottom flange must be of the same area as the tes joined, and of sufficient length to take a number of rivets equal to the ngth of the cover-plate.

3. Design of Plate and Box Girders

The Principal Steps in the Design of Riveted Girders. In designing iveted girder to sustain safely a given load, the following steps are necessary:
(1) The determination of the required flange-area.
(2) The determination of the thickness of the web to resist
 (a) Shearing.
 (b) Buckling. This step also determines if stiffeners are necessary.
(3) The determination of the number and pitch of the rivets.
(4) The approximate weight of the girder.
(5) The determination of the length of the flange-plates when more than one equired for each flange.

(1) **The Flange-Area.** In determining the FLANGE-AREA of riveted girders, is customary to assume that the bending moments are entirely resisted by the er and lower flanges, the web being assumed to resist the shear only. Just t should be included in the flange-area is a question on which engineers differ. e include the flange-plates and angles and one-sixth of the web-area, others flange-plates and angles only, while others include the flange-plates and only horizontal legs of the angles, the vertical legs being considered as belonging the web. In compression-flanges, usually the upper ones, the gross section- may be taken, provided the rivets are machine-driven and fill completely holes; but in tension-flanges, usually the lower ones, the net area is taken, t is, the gross area minus the area of the greatest number of rivet-holes in cross-section, since the stresses of tension are not transmitted through the s as are those of compression.
A general FORMULA* FOR DETERMINING THE FLANGE-AREA, which applies to conditions of loading is

$$\frac{\text{Area of one flange}}{\text{in square inches}} = \frac{\text{maximum bending moment in foot-tons}}{\text{depth of web in feet} \times \text{safe unit fiber-stress in tons}}$$

$$A = M_{max}/dS \qquad (1)$$

* This may be derived from what is sometimes called the PLATE-GIRDER FORMULA, $M_{max} = SAd$, in which S is the safe unit bending-stress in the flange at the section of mum bending moment, A is the area of the cross-section of either flange and d is ap-ately the depth of the girder. Of course the units must be the same in both mem-of the equation. If the center of moments is taken at the center of gravity of the s-section of either flange-area and if the area of metal resisting bending is considered concentrated in the flanges, the depth of each being very small compared to that e girder-depth, then SA is the total horizontal stress in either flange, d its lever-and SAd the resisting moment of the cross-section, equal to M_{max}. Hence $A = M_{max}/dS$. Another method uses the section-modulus, $I/c = M/S$, in determining the e-areas and proportioning the girder. (See pages 706 to 716.)

Rules for finding the MAXIMUM BENDING MOMENT for different conditions loading are given in Chapter IX.

S, the SAFE UNIT FIBER-STRESS FOR FLANGE-BENDING, was formerly taken from 13 000 to 16 000 lb per sq in, the tables in the manufacturers' handbo giving the safe loads, etc., for riveted girders, varying in regard to this stress.*

If it is required to compute the SAFE UNIFORMLY DISTRIBUTED LOAD fo girder already constructed or designed, the following formula † may be u The safe load in pounds, uniformly distributed is

$$W = \frac{8 \times \text{net area of bottom flange} \times \text{depth in ft} \times S}{\text{span in feet}}$$

or

$$W = 8\,AdS/l$$

From the result the weight of the girder itself should be subtracted.

For the SAFE CONCENTRATED LOAD AT THE MIDDLE OF THE SPAN take one- the result obtained by formula (2) and subtract the weight of girder. Case IV, page 326.)

(2) **The Thickness of the Web.** The thickness of the web is determ by its resistance to VERTICAL SHEARING. Whether or not stiffeners shall be is determined by the resistance of the web to BUCKLING.

(a) **Shearing.** To resist the vertical shear the NET SECTIONAL AREA OF WEB in square inches must be

$$A = \frac{\text{the maximum vertical shear}}{S}$$

or

$$A = V_{max}/S$$

V and S being both in tons, S is taken at 10 000 ‡ lb or 5 tons per sq in. Table II, page 703.)

The MAXIMUM VERTICAL SHEAR in any beam or girder is at the greater rea and is equal to it.

For a girder supported at both ends and uniformly loaded with a loa the MAXIMUM VERTICAL SHEAR is

$$V_{max} = W/2$$

For a girder supported at both ends and loaded at the middle with a lo

$$V_{max} = P/2$$

For a girder supported at both ends and loaded as in Fig. 7,

$$V_{max} = Pm/l = R_1$$

For a girder supported at both ends and loaded with two equal concent loads P, P, equally distant from the middle, as in Fig. 8,

$$V_{max} = P = R_1 = R_2$$

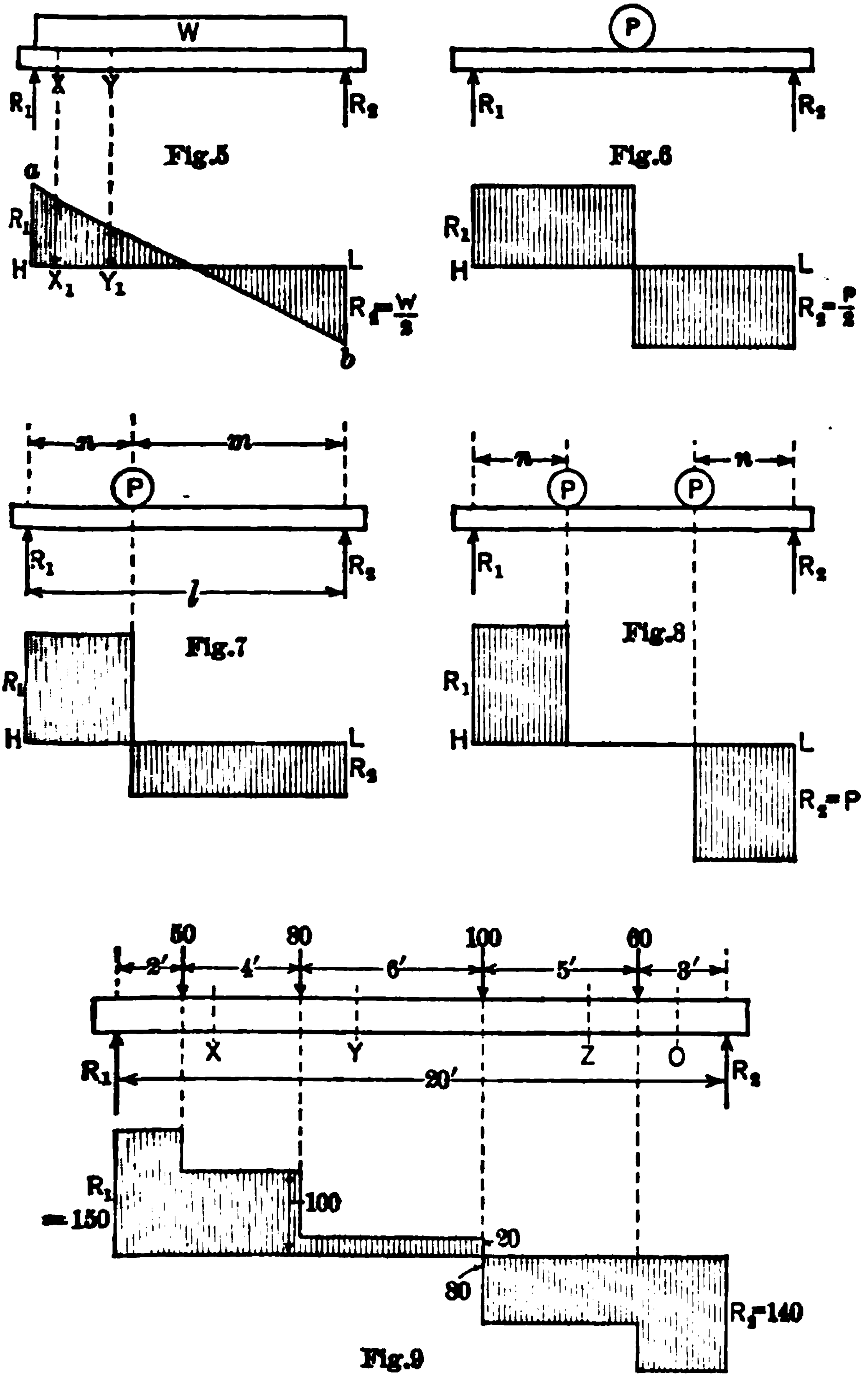
For combinations of loads the maximum vertical shear will equal the g reaction. The method of determining the REACTIONS at the supports of a or girder is given in Chapter IX, Subdivision 1. The VERTICAL SHEA

* See Chapter XV, paragraphs relating to riveted single and double-beam gird foot-note with same, pages 603 and 604; also page 704. The value in most city now 16 000 lb per sq in.

† From Formula (1) just explained, and from Case V, page 326, $M_{max} = SA$ $M_{max} = Wl/8$. Hence $Wl/8 = SAd$ and $W = 8\,AdS/l$.

‡ This is a conservative value. The Carnegie Pocket Companion and the b laws of most cities permit 10 000 lb per sq in for steel.

ICE at any given vertical section of a beam or girder between the supports is e algebraic sum of all the vertical external forces acting on the beam to the



Figs. 5 to 9. Diagrams for Vertical Shears for Different Loadings

f that section, forces acting upwards being considered as plus, and those g downwards being considered as minus.

s, in the case of the beam shown in Fig. 9, the REACTION R_1 will be l, by the method explained in Example 2, page 323, and by Formulas (2)

and (3), page 323, to be 150 lb, and that at R_2 to be 140 lb. The shear at v
ous sections may be found by applying the foregoing definition of VERTI
SHEAR, thus:

Shear at $X = +150 - 50 = +100$ lb
Shear at $Y = +150 - 50 - 80 = +20$ lb
Shear at $Z = +150 - 50 - 80 - 100 = -80$ lb
Shear at $O = +150 - 50 - 80 - 100 - 60 = -140$ lb

The manner in which the VERTICAL SHEAR varies between the supports, u
different dispositions of the loads, is shown graphically by the hatched are
Figs. 5 to 9; in the first three cases W and P are assumed to have the s
value.

When the load is UNIFORMLY DISTRIBUTED the VERTICAL SHEAR can be fo
graphically by laying off vertically R_1 and R_2 to a scale of pounds, and dra
the line ab, Fig. 5. The shear at X will then be represented by the ordi
X_1 and the shear at Y by Y_1, and they can readily be scaled.

(b) Buckling.* The safe resistance of the web to BUCKLING, in pounds
square inch, may be determined by the formula

$$S_b = \frac{10\,000}{1 + \dfrac{d^2}{3\,000\,t^2}}$$

in which S_b is the safe buckling value in pounds per square inch, d is the d
of the web in the clear between flange-plates in inches and t is the thickne
the web in inches. When this resistance is less than the UNIT STRESS
VERTICAL SHEAR at any section, stiffeners must be used.

Stiffeners. These should be made of ANGLES, not less than 3½ by 3½ b
in in size. They should always be tightly fitted between the flange-angle
as to support the horizontal flanges.
order to bring the stiffeners in contact
the web and the vertical leg of the an
FILLERS, of the same thickness as
flange-angles, are generally used, as sh
in Fig. 10. Where there are several gi
exactly alike, something may be save
omitting the fillers and BENDING THE
FENERS, as shown in Fig. 11. This b
ing, however, can be done properly,
by the use of special dies, and costs
than the fillers unless there are m
stiffeners. The SPACING OF STIFFENE
more a matter of judgment and expe
than of exact calculation. SHEAR-DIAG
shown in Figs. 5 to 9, are of great assistance in visualizing shearing-str
The general rule is to place the stiffeners not farther apart than the dep
the full web-plate on girders over 3 ft in depth, with a maximum spacing of
On girders under 3 ft in depth they are placed 3 ft apart. Girders 2 ft an
in depth require no stiffeners. On girders supporting distributed loads the
generally placed nearer together at the ends than towards the middle.

Fig. 10. Stiffeners with Fillers

Fig. 11. Bent Stiffeners

* See Table III, page 705, and also in Chapter XV, the paragraphs and foot
pages 567 to 569, relating to the web-buckling of beams and girders. The fo
used for web-buckling in Table III, page 705, is the formula that was used in the B
Steel Company's Manual, and as the values computed by it vary but little from
deduced by the Cambria formula (see page 568), Table III is retained as it is.

Stiffeners should always be placed at the ends of girders and directly over the edge of each support, as shown in Fig. 18, and wherever there are concentrated loads. On plate girders the stiffeners are always placed on each side of the web; on box girders on the outside only.

The Bearing of Girders. This depends somewhat upon the character of the loading, but a safe general rule is to make the BEARING of the girder beyond the edge of the support equal to ONE-HALF THE DEPTH OF THE GIRDER.

(3) The Number and Pitch of the Rivets. **(a) Rivets in Web-Legs of Angles.** It will readily be seen that when a plate or a box girder is loaded, the tendency of the bending moments is to cause the flange-plates and angles to SLIDE HORIZONTALLY past the web; this tendency is resisted by the rivets which connect the angles with the web. The TOTAL AMOUNT OF THIS TENDENCY TO SLIDE, called the HORIZONTAL FLANGE-STRESS, between any section of the flange and the nearer end of the girder, is equal to the BENDING MOMENT at that point DIVIDED BY THE DEPTH OF THE WEB.* The TOTAL NUMBER OF RIVETS between that section and the nearer end must be such that their combined resistance to SHEARING or BEARING, whichever has the lower value, shall equal this horizontal flange-stress at the section; or

$$\text{number of rivets} = \frac{\text{horizontal flange-stress}}{\text{bearing or shearing of one rivet}} \qquad (5)$$

and the total number of rivets in the web-angle from end to end is twice this, or

$$\text{total number of rivets} = \frac{2 \times \text{maximum bending moment}\dagger \text{ in foot-pounds}}{\text{depth of web in feet} \times \text{least resistance of one rivet}} \qquad (6)$$

If the NUMBER OF RIVETS determined by formula (6) is such that they would be more than 6 in apart, then the number must be increased, as in no case should they have a greater PITCH than 6 in.

(b) Rivets in Flange-Legs of Angles. WITH A SINGLE COVER-PLATE. For girders with a single cover-plate, it is customary to put the same number of rivets in the flange-leg as in the web-leg for a distance of 3 ft from the ends of the girder, STAGGERING the rivets as in Fig. 15. Beyond that point to the middle of the girder one-half the number of rivets will be sufficient, provided this will not give them a greater pitch than 6 in.

WITH TWO OR MORE COVER-PLATES. When two or more cover-plates are used, each plate must have sufficient rivets between the end of the plate and the point where its resistance is required, that is, for example, between a and b, Fig. 13, to transfer to the angle and flange-plates between, an amount EQUAL TO THE SAFE STRENGTH OF THE PLATE. From this point to the middle point of the girder, the rivets can be spaced according to the rule for the greatest pitch.

(c) Rivets in Stiffeners. The spacing of rivets in the stiffeners is generally determined by the rules given for the pitch of rivets. Further explanation of the method of determining the spacing of rivets will be found in the following examples.

(4) The Approximate Weight of the Girder. In determining the size of a riveted girder to support a given load, it is desirable to be able to add to the

* See Formula (1), page 683, and foot-note relating to it. $M = SAd$, and hence $SA = M/d$, SA being the total amount, in pounds, of the tendency to slide, and S being the horizontal unit, flange, fiber-stress in pounds per square in, due to flexure. A is the area in square inches of the cross-section of the flange and d is the approximate depth of the girder.

† Because the maximum horizontal flange-stress is equal to the maximum bending moment divided by the girder-depth, or $S_{max}A = M_{max}/d$.

superimposed load the WEIGHT OF THE GIRDER itself, as this often forms a c
siderable part of the load to be supported. The following empirical rule
often used to determine the approximate weight of a plate or box girder:

$$\text{Weight of girder between supports, in tons} = Wl/700$$

in which W equals the load to be supported, in tons, and l equals the spa
feet. The constant 700 was determined for girders of from 35 to 50 ft long,
may be used without much excess for girders of shorter spans.

(5) The Determination of the Lengths of the Flange-Plates. Fo
methods used to determine these, see the following examples.

4. Explanation of Tables

The Calculations for the Design of Riveted Girders may be gr
facilitated by the use of Tables I, II, III and IV at the end of this chapt

Table I gives the sectional area that should be deducted for rivet-hol
plates of different thicknesses. In computing this table 1/8 in was added t
diameter of the rivet to allow for the injury to the metal caused by pun
and also to allow for the expansion of the heated rivet.

Table II gives the safe shearing value for web-plates for various depths
thicknesses, and the deduction to be made for each 3/4-in or 7/8-in rivet.

Table III gives the safe resistance to buckling per square inch of net sec
and also the total safe resistance in pounds for the more common sizes of
plates, with two rivet-holes deducted. It is very seldom that any vertica
tion between the stiffeners contains more than two rivet-holes. Tables g
the dimensions and properties of angles will be found in Tables XI and
pages 362 to 367, and the shearing value and bearing values of rivets are
in Tables II and III, pages 418 and 419.

Table IV gives the elements of riveted plate girders of various depths,
which it is possible to select economical sections for almost any ordinary c
tion of loading.

5. Examples of Plate and Box Girders

Example 1. It is required to support the floor over a room 50 by 6
means of riveted steel plate girders, placed across the room, 16 ft on
The room above is to be used for general assembly purposes. The flo
are of wood and there is a plaster ceiling on the under side of them. Th
of the girder is required.

First Step. The Load. The first step is to determine the load to be su
by each girder. The floor-area supported by each girder is 50 by 16 ft, o
ft The weight of the floor-construction between the girders will not
25 lb per sq ft, and an allowance of 100 lb per sq ft for the live load
ample. The unit load, 125 lb × 800 = 100 000 lb, or 50 tons, the lo
carried by the girder. To this should be added the weight of the gird
Substituting in Formula (7),

the approximate weight of the girder $= \dfrac{50 \times 50}{700} = 3.57$ tons, or about
and the total load, in round numbers, is 107 000 lb. This, of cours
formly distributed.

* From "Compound Riveted Girders," by W. H. Birkmire.

Second Step. The Flange-Area. The next step is to determine the flange-ea. Before this can be done, however, the width and depth of the girder st be decided. As it is desirable to keep the girder as shallow as possible, nsistent with good engineering, the case will be considered an exceptional e and the depth of the web-plate will be made 36 in. which is about one-teenth the span and a little less than the usual limit.

As the girders are braced sidewise by the floor-joists, it will not be necessary make the width of the flange-plates one-twentieth the span of the girder, d it may be made 12 in width. The flange-area may be determined by rmula (1), page 683, and is

$$A = M_{\max}/dS$$

$$\text{flange-area (sq in)} = \frac{\text{maximum bending moment (ft-tons)}}{\text{depth of web (ft)} \times S \text{ (tons per sq in)}}$$

The maximum bending moment for a uniformly distributed load on a simple m is $M_{\max} = Wl/8$ (Case V, page 326), or in this particular case, 53.57 s × 50 ft/8 = 334.8 ft-tons.

The value of S * has varied in the handbooks from 6 to 8 tons, depending n varying conditions and upon the judgment of engineers. A value of S tons or 16 000 lb per sq in is the requirement of the new New York Building de, and of the codes of most cities. In this example 14 000 lb per sq in is med for S.

bstituting this value in the formula gives for the net area of either flange, 8/(3 × 7) = 16 sq in.

he upper flange may now be designed. For a girder of this size and loaded is way, it will be advisable to try two 5 by 3½ by 7/16-in angles, with the legs horizontal.† The sectional area of these angles (Table XI, page 363) 06 sq in which leaves 9 sq in for the area of the flange-plates. Dividing this 2-in, the width of the flange, gives ¾ in for the total thickness of the plates, h may be made up of two ⅜-in thick plates. Of course, any other combina- of plates and angles having an area of cross-section of 16 sq in will fulfill conditions of the problem, the selection in all cases depending upon the ment and experience of the designer. Note, also, that no part of the web een included in the flange-area although it would be safe to include one- of it. This also is a matter of individual opinion.

the lower flange is in tension, the rivet-holes should be deducted in order tain the net area. Assuming that ¾-in diameter rivets are used, it will ted that the greatest loss of section is by two rivet-holes opposite each connecting the angles with the plates of the bottom flange. From Table I, 02, the area of two ¾-in rivets in a ¾-in plate is 1.31 sq in, and in a plate, the same thickness as that of the angles, it is 0.76 sq in. The sum e thicknesses is 2.07 sq in, which must be added to the net area of the flange-plates, 16 sq in, making 18.07 sq in for the gross area of the lower

, in Chapter XV, paragraphs and foot-notes, page 603, relating to fiber-stresses ling for riveted beam girders, etc.

the flange-angles of plate girders the 5 by 3½-in size is most commonly used, flange-plate is 12 in wide, and 6 by 4-in angles when the flange-plate is over 12 in For box girders 5 by 4, 5 by 3½, 4 by 3½ and 3½ by 3½-in are common sizes; very heavily loaded girders, requiring two rows of rivets in the web-leg, 6 by s are often used. For most riveted girders, in which only one row of rivets is the short leg is riveted to the web, so as to bring most of the material as far from ral axis of the girder as possible. The minimum thickness of flange-angles ⅜ in, and the maximum thickness for ordinary loads is ¾ in.

flange-plates. This additional area may be obtained by increasing the thi ness of the plates to ½ in.

The flanges will then be made up as follows:

Upper flange:	Two angles 5 by 3½ by 7/16-in	= 7.06 sq in, gross area
	Two plates 12 by ⅜-in	= 9.00 sq in, gross area
	Total	16.06 sq in, gross area
Lower flange:	Two angles, 5 by 3½ by 7/16-in	= 7.06 sq in, gross area
	Two plates, 12 by ½-in	= 12.00 sq in, gross area
	Total	= 19.06 sq in, gross area

Third Step. The Length of the Flange-Plates. To determine this it necessary to plot the bending-moment diagram shown in Fig. 12. The bendin

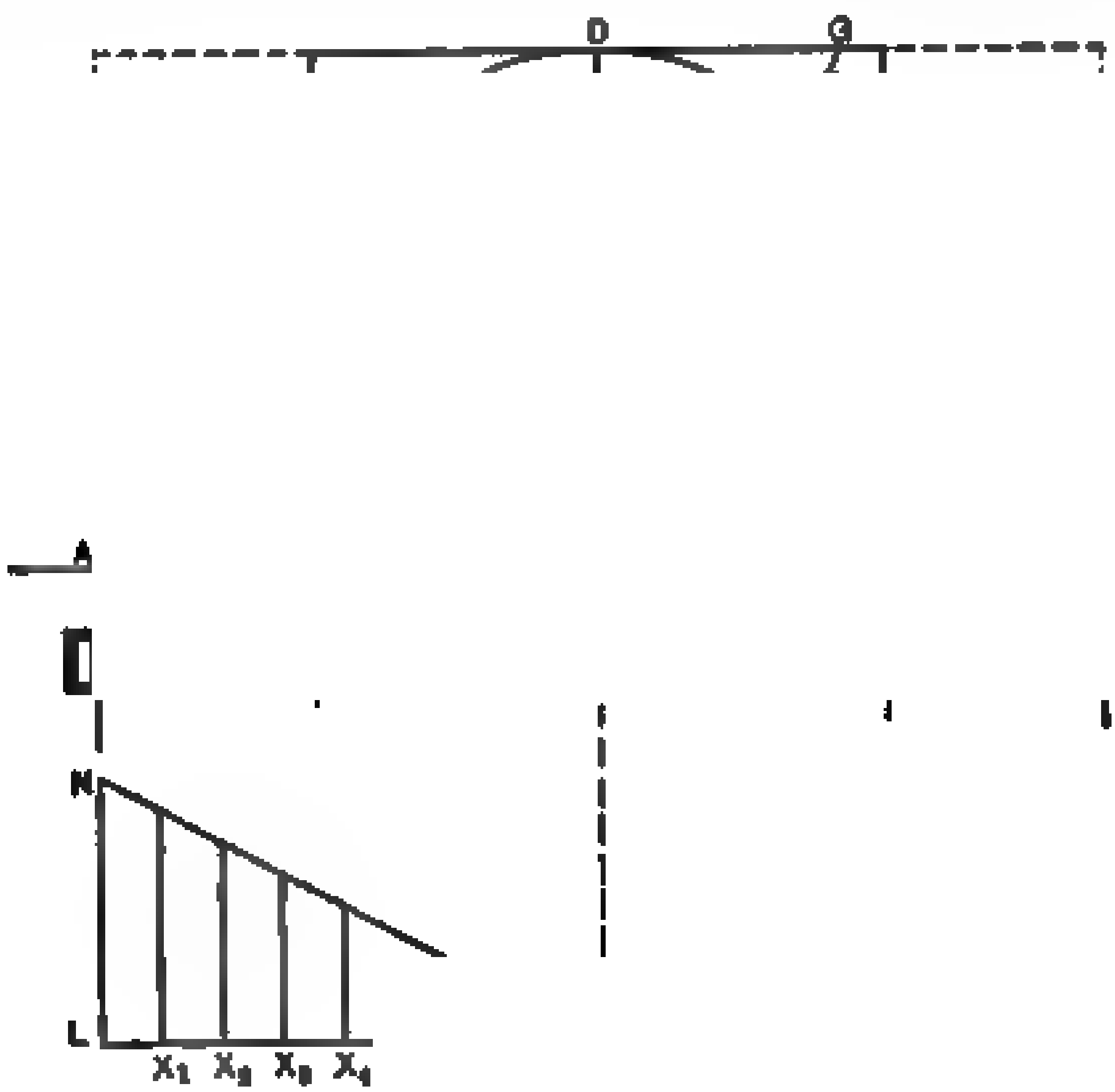


Fig. 12. Diagrams for Bending Moments and Vertical Shears. Example 2

moment diagram for a girder under a uniformly distributed load is bounded a parabola having a height over the middle of the girder equal to the maximu bending moment. From the middle point *C*, of a horizontal line *AB*, at a convenient scale, lay off a vertical line *CD*, equal to the maximum bend moment, 334.8 ft-tons. Construct the parabola *ADB* (see page 79); then bending moment at any other point, as *E*, is equal to the ordinate *EF* at that point, measured to the same scale.

To find the theoretical length of the flange-plates of the lower flange, inclose the bending-moment diagram in a rectangle and from any convenient point, such as C, lay off any line CG, equal to the total flange-area, 19 units in length, and at such an angle that the upper end G will lie on a horizontal line drawn through D. Divide the line CG into three parts, CH representing the sectional area of the angles, equal to 7 units, and HI and IG representing the sectional area of the two plates, equal to 6 units each. Draw horizontal lines through H and I; then the line JJ will represent the theoretical required length of the second or upper flange-plate and the line KK the length of the first or lower flange-plate. In practice, however, the plates are usually extended beyond the points J and K on each side as an additional factor of safety, a distance sufficient to take enough rivets to transmit at least one-third the resistance of the plate. It is also customary to make the first or lower flange-plate the full length of the girder as it greatly stiffens the angles and adds but a small amount to the cost. Theoretically the length of the flange-plates of the top flange would be less than the length of the plates of the lower flange, because the flange-area of the top flange is less than that of the lower flange; but they are usually made the same length.

Fourth Step. The Web. Webs are proportioned to resist the shear. The maximum shearing-stress in a girder uniformly loaded is equal to either reaction, which in this case is one-half the total load, or 53 500 lb. As the girder is 3 ft deep, this small shear would require a very thin section, thinner than the minimum thickness for webs, which is ⅜ in. From Table II, page 703, it is seen that the shearing resistance of a ⅜ by 36-in web-plate is 135 000 lb, which is greatly in excess of the actual shear.

Fifth Step. The Stiffeners. As before explained, stiffeners will be required whenever the vertical shear exceeds the safe resistance of the web to buckling. The vertical shear is 53 500 lb and the resistance to buckling may be found from Table III, page 705. This, for a ⅜ by 36-in web with two ¾-in rivets is found to be 31 560 lb; hence stiffeners will have to be used. As stated under Buckling of Web, page 686, the spacing of stiffeners is more a matter of judgment and experience than of exact calculation, and for this a shear-diagram, also shown in Fig. 12, is of great assistance. It may be constructed as follows: On a horizontal line LM, lay off to any convenient scale vertical lines LN and MP, each representing 53 500 lb. Connect the points N and P; then the vertical shear at any point is equal to the ordinate at that point, measured to the same scale. Thus, at X_1, 3 ft from the left end, the shear is 47 500 lb, at X_2, 6 ft from L, it is 40 500 lb, at X_3, 9 ft from L, it is 34 000 lb and at X_4, 12 ft from L, it is 27 500 lb. As the vertical shear at X_3 is greater than the safe resistance to buckling and at X_4 less, it might be safe to stop the stiffeners at X_4; but as the floor-joists are framed flush, or nearly so, with the top of the girder, and rest upon angles riveted to its web, it will be advisable to put about 3 stiffeners between X_4 and the corresponding point on the right-hand half of the girder. Additional stiffeners should be placed directly over each support, making 15 stiffeners on each side of the girder. These will be made of 4 by 4 by ⅜-in angles.

Sixth Step. The Number and Pitch of the Rivets. First, the number of rivets in the web will be considered. As a rivet is required at the end of each stiffener, it will be necessary to determine the number and spacing of the rivets between each pair of adjacent stiffeners. In the web, the rivets are in double shear. In Tables II and III, pages 418 and 419, values are given based upon unit shearing values of 7 500 and 10 000 and bearing values of 15 000 and 20 000 lb per sq in. (See foot-notes with these tables.) The shearing resistance

of a ¾-in rivet at 10 000* lb per sq in is 4 420 × 2 = 8 840 lb for double she and the bearing value of the same rivet in a ⅜-in plate, at 20 000* lb per sq is 5 630 lb. As the bearing value is the smaller, it will determine the num of rivets required.

The number of rivets from either end of the girder to any point depends u the horizontal flange-stress at that point, and it has been shown that the flan stress is equal to the bending moment divided by the depth of the web. (foot-notes with Equations (5) and (6).) Scaling off the bending moment ab the point X_1 gives 75 ft-tons; hence the horizontal flange-stress is equal to 75/. 25 tons = 50 000 lb. The number of rivets required between this point and left reaction is, from Formula (5), equal to 50 000/5 630 = 10 rivets, which to be spaced in a distance of 36 in, making the spacing 3.6 in. Above X_2 bending moment scales 141.24 ft-tons, the flange-stress is 141.24/3 = 47.08 t or 94 160 lb, and the number of rivets required between X_2 and A is 94 160/5 = 17; but 10 of these are required between X_1 and A, leaving 7 to be pla between X_1 and X_2 in a distance of 36 in making the spacing 5.1 in. At X_3, bending moment scales 197.4 ft-tons, and the flange-stress is 197.4/3 = 65.3 t or 130 600 lb. The number of rivets required is 130 600/5 630 = 24, but 1 these are required between X_2 and A, leaving 7 to be placed between X_2 X_3, making the spacing the same as in the second panel. At X_4 the benc moment scales 243.96 ft-tons, and the flange-stress is 243.96/3 = 81.32 ton 162 640 lb. The number of rivets required is 162 640/5 630 = 30, but 2 these are required between X_3 and A, leaving 6 to be placed between X_3 and in a distance of 36 in, making the spacing 6 in. As this is the maximum spa allowed, it will be used from X_4 to the corresponding point on the oppo right-hand half of the girder. The same number of rivets will be used in flange-legs of the angles as in the web-legs, but they will be spaced so t they will come between those in the web.

The outer flange-plate scales 28 ft 6 in in length in the bending-moment gram, but this length, as before stated, should be increased sufficiently to t enough rivets to transmit at least one-third of the resistance. The area of plate is ½ in × 12 in = 6 sq in, minus the area of two ¾-in rivet-holes, 0.8 in (Table I, page 702), leaving a net area of 5.13 sq in. The resistanc the plate is therefore equal to 5.13 sq in × 14 000 lb per sq in = 71 820 lb. (third of this, or 23 940 lb, must be transferred by rivets placed beyond points JJ. As the rivets in the flange are in single shear, the shearing v of one rivet in single shear, 4 420 lb, will govern. The number of rivets quired, then, is 23 940/4 420 = 6, or 3 in each angle. The spacing of the ri in this panel is 6 in. The plates will therefore be extended 18 in on either of JJ.

* The shearing value of rivets is taken at from 7 000 to 12 000 lb per sq in an bearing value at from 12 000 to 24 000 lb per sq in. The usual values are 10 000 l shear and 20 000 or 24 000 lb for bearing. Values of 12 000 lb for shear and 24 0 for bearing are the requirements of the New York Building Code A bearing other than those of Tables II and III, pages 418 and 419 is purposely used in this exa as it is frequently necessary to use different unit stresses than those from which particular table has been computed. If no other table is at hand for the values based some particular rivet bearing-stress. Tables II and III, pages 418 and 419, be used and the new value found by proportion; or the bearing-stress ca found by multiplying the product of the diameter of the rivet and the t ness of the web by the new unit stress. In this example, Table III, page gives, for 18 000 unit stress, 5 060 lb for bearing; 10/9 of this gives 5 6 for a 20 000 unit stress. Also, ⅜ in by ¾ in by 20 000 lb per sq in = lb.

Splices. As the total length of the girder is but 53 ft, it will not be necessary to splice the webs or the flanges, because the extreme length of a ⅜ by 36-in plate is 110 ft and of a 12 by ½-in plate, 90 ft.* It is never necessary to splice angles as they are rolled in lengths up to 90 ft. In very long, deep girders, however, it is sometimes necessary to splice the web, and the joint is sometimes made at the middle, as theoretically there is no vertical shearing-stress in the web

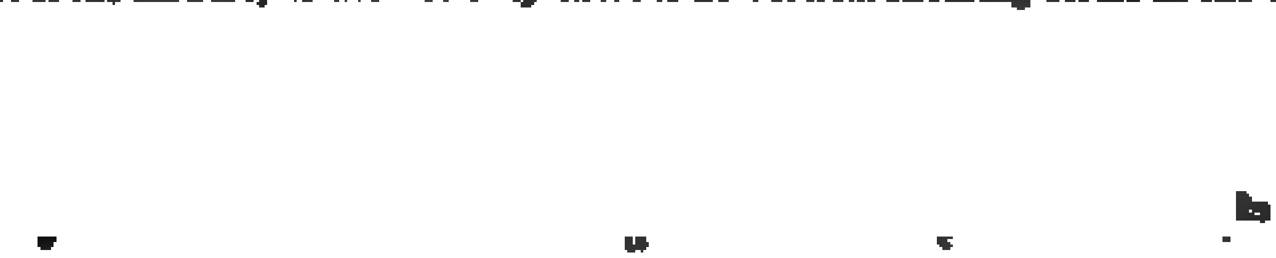


Fig. 13. Splicing of Inner Plate of Bottom Flange of Plate Girder. Example 1

at that point when the load is uniformly distributed. Generally, however, the web is spliced in two places, equidistant from the middle of the girder. The splice is calculated for vertical shear only, the rule being to divide the shear at the splice by the safe shearing value or bearing value of one rivet. This gives the number of rivets required on each side of the splice-plate, unless the maximum pitch is exceeded, when more are added.

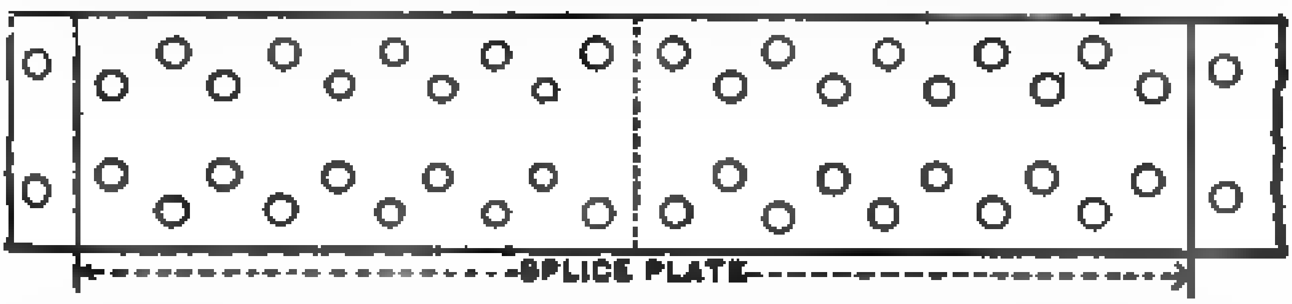


Fig. 14. Plan of Splice-plate. Example 1

Whenever a splice is required in a flange-plate, it should be, if possible, at a point just beyond the end of the plate above it. The joint must be made by fitting to the spliced plate, a plate of the same thickness and of sufficient length to receive a number of rivets on each side of the joint equal to the strength of the plate that is spliced. When the flange is made up of two plates of the same thickness, the simplest method of splicing the inner plate is as shown in Fig. 13.

Let *e* denote the theoretical position of the end of the outer plate, as determined by the bending-moment diagram, and *a* the point to which the plate must be extended to receive rivets of a resistance equal to one-third the strength of the plate. Then let the joint in the inner plate be just over *a* and extend the splice-plate to *b*, or such a distance that it can receive a number of rivets equal in resistance to the strength of one plate.

* Tables of extreme lengths are published in the various handbooks. The above dimensions, for example, are taken from the table on page 111 of Carnegie's Pocket companion.

Fig. 15 shows one end of the girder, drawn according to the foregoing calcu-lations.

The Bill of Quantities for the Girder. The following is a bill of quantitie for the construction of thi girder.

Load: 100 000 lb, uniforml distributed. Span 50 ft Depth 3 ft

Upper flange:
- Two angles, 5 by 3½ b 7/16 in, 53 ft long
- One plate 12 by ¾ in, 5 ft 0 in long
- One plate, 12 by ¾ in, 3 ft 6 in long

Lower flange:
- Two angles, 5 by 3½ b 7/16 in, 53 ft long
- One plate, 12 by ½ in, 5 ft 0 in long
- One plate, 12 by ½ in, 3 ft 6 in long

Web:
- One plate, 36 by ⅜ in, 5 ft 0 in long
- 30 stiffeners, 4 by 4 b ⅜-in angles, 2 ft 11 l long
- 30 filler-plates, 4 by ¼ i 29 in long
- 92 ft 8 in of 4 by 4 b ½-in angles for sup porting floor-joists

Rivets:
- ¾ in in diameter

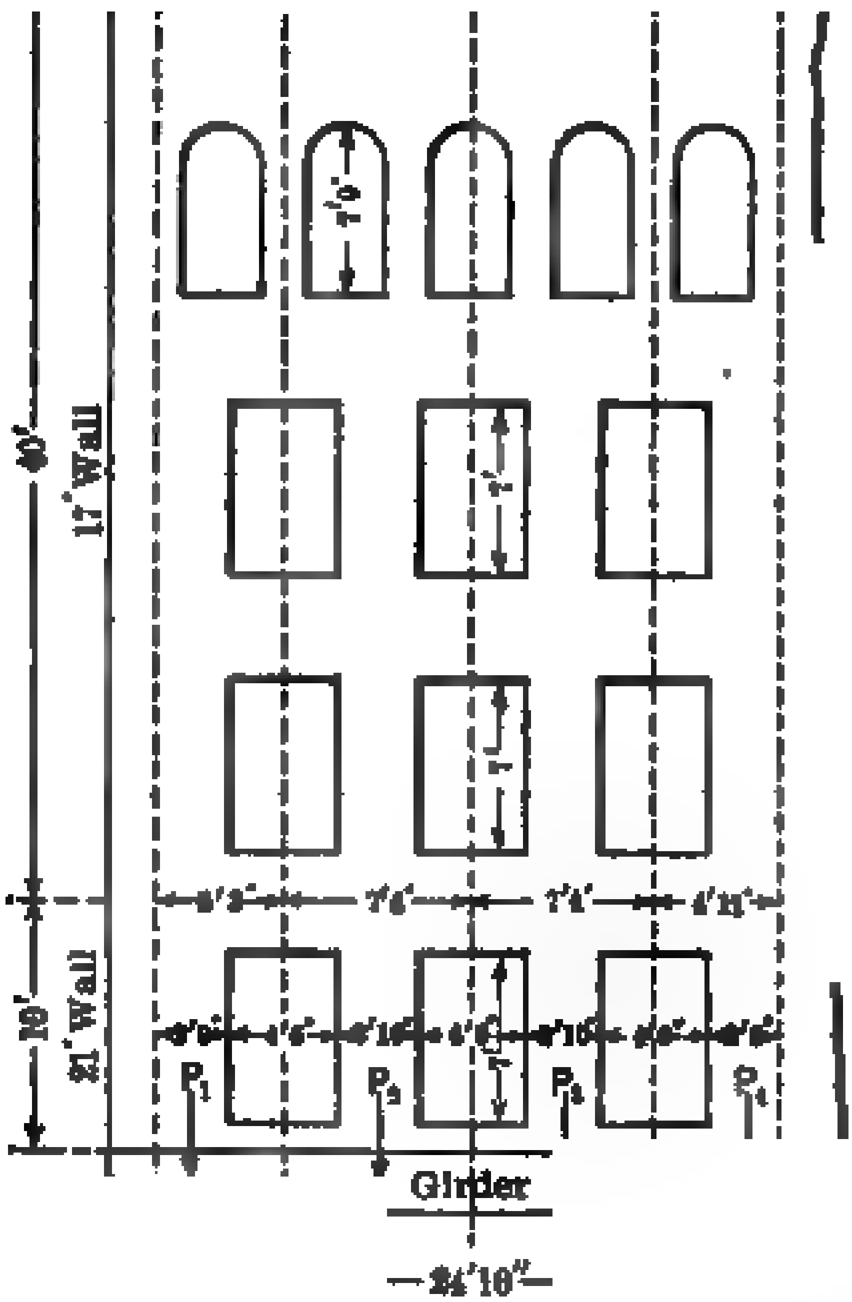


Fig. 16. Box Girder Supporting Brick Wall. Example 2

Example 2. The wall show in Fig. 16 is to be supported b a riveted-steel box girder at th height indicated. It is re quired to design the girder.

First Step. The Load. The first step towards designing the girder is the de termination of the load. The space under the lower windows is too small to dis tribute the weight from the piers uniformly over the girder, so that the only sa assumption is that the weight of the wall between the lines *A* and *B* is conce trated at P_1, the weight of wall between lines *B* and *C* at P_2 and so on. Th floor-joists run across the building, so that only the weight of the wall will b supported by the girder. Allowing 200 lb per square foot of face for the 21- wall, and 165 lb for the 17-in wall, both walls being plastered on the inside:

Load at P_1

$$= \left\{ \begin{array}{l} [5'\,3'' \times 10' - 7' \times 2'\,3''] \times 200 = \ldots\ldots\ldots\ldots\ldots\ldots\ 7\,350 \\ [5'\,3'' \times 40' - (2'\,3'' \times 14' + 3'\,2'' \times 7')] \times 165 = \ldots\ 25\,795 \end{array} \right\} = 33\,145$$

nd at P_2

$$\left\{\begin{array}{l}[7'\,4''\times 10' - 4'\,6''\times 7'\,0''] \times 200 = \ldots\ldots\ldots\ldots\ldots\ 8\,366 \\ [7'\,4''\times 40' - (4'\,6''\times 14' + 4'\,9''\times 7')]\times 165 = \ldots\ldots 32\,354\end{array}\right\} = 40\,720\text{ lb}$$

nd at P_3 = that at P_2 = ... 40 720 lb

nd at P_4

$$\left\{\begin{array}{l}[4'\,11''\times 10' - 2'\,3''\times 7'] \times 200 = \ldots\ldots\ldots\ldots\ldots\ 6\,683 \\ [4'\,11''\times 40' - (2'\,3''\times 14' + 3'\,2''\times 7')]\times 165 = \ldots 23\,595\end{array}\right\} = 30\,278\text{ lb}$$

Total load on girder = ... 144 863 lb

72.4 tons

m Equation (7)

pproximate weight of girder $= \dfrac{W \times l}{700} = \dfrac{72.4 \times 24\frac{5}{6}}{700} = 2.5$ tons, or 5 000 lb

ut one-third of this, or say 1 600 lb, should be added to P_2 and P_3, and 900 lb P_1 and P_4. This will give, approximately, the following loads, applied as in . 17:

$P_1 = 34\,000$ lb $\qquad P_2 = 42\,300$ lb

$P_3 = 42\,300$ lb $\qquad P_4 = 31\,200$ lb

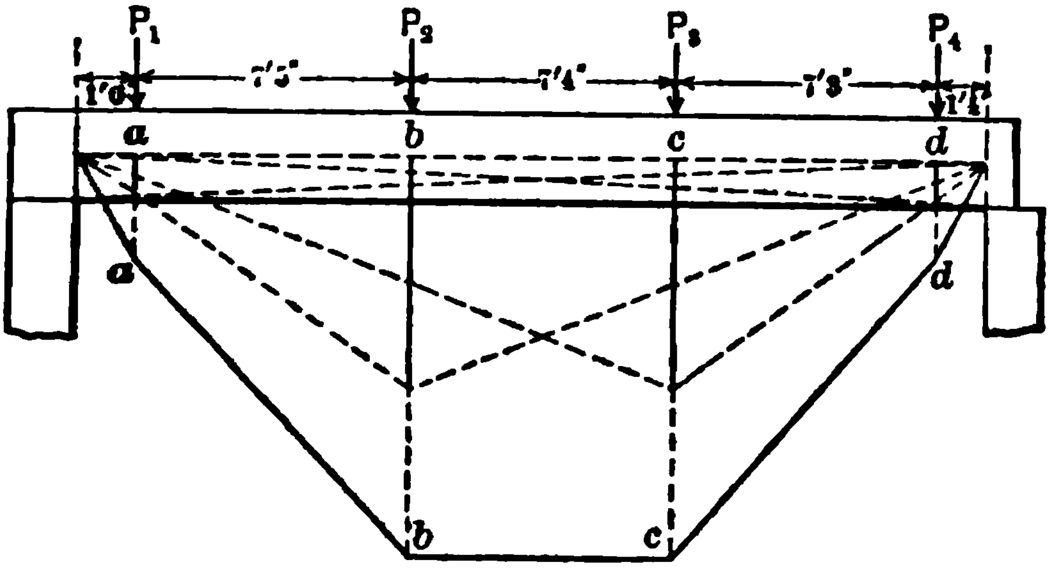


Fig. 17. Diagram for Bending Moments. Example 2

cond Step. The Determination of the Maximum Bending Moment. By s of the formula under Case VI, page 327, the maximum bending moment t-pounds for the loads are found to be as follows:

$$\text{For } P_1,\ M_{\max} = \frac{34\,000 \times 1'\,6'' \times 23'\,4''}{24'\,10''} = 47\,980 \text{ ft-lb}$$

$$\text{For } P_2,\ M_{\max} = \frac{42\,300 \times 8'\,11'' \times 15'\,11''}{24'\,10''} = 242\,000 \text{ ft-lb}$$

$$\text{For } P_3,\ M_{\max} = \frac{42\,300 \times 16'\,3'' \times 8'\,7''}{24'\,10''} = 237\,900 \text{ ft-lb}$$

$$\text{For } P_4,\ M_{\max} = \frac{31\,200 \times 1'\,4'' \times 23'\,6''}{24'\,10''} = 39\,420 \text{ ft-lb}$$

ting these moments to a scale, as explained for Fig. 15, page 329, the ng-moment diagram shown in Fig. 17 * is obtained. The maximum bend-

he bending moments in this diagram are drawn to a scale of about 400 000 ft-lb ch.

ing moment is at P_2, over the longest ordinate bb and where the vertical s is zero, and is equal to the length of the ordinate bb, which scales 418 000 or 209 ft-tons.

Third Step. The Determination of the Flange-Area and the Length of Cover-Plates. Before these can be determined, the depth of the web-must be decided. As there is nothing to limit the depth of the girder, it be made about one-tenth of the span, or 30 in. Then by Formula (1), 683, $A = M_{max}/dS$, and using 14 000 lb or 7 tons per sq in for S,

$$\text{the gross area of upper flange} = 209/2.5 \times 7 = 12 \text{ sq in}$$

As the thickness of the wall to be supported is 21 in, the flange-plate mu at least 20 in wide and not less than ⅜ in thick. The sectional area of by 20-in plate is 7½ sq in, leaving 4½ sq in to be made up by the a The sectional area of two 5 by 3½ by 7/16-in angles is 7.06 sq in (Table page 363), which leaves a small excess for the lower flange. For rivets in diameter, the loss in area due to two rivet-holes in a ⅜-in plate is (Ta page 702) 0.65 sq in and in a 7/16-in plate, the thickness of each angle, 0.76 making 1.41 sq in in all, for which the excess in the angles is more than cient. The width of the flange being more than one-twentieth the span m lateral support unnecessary.

Fourth Step. The Webs and Stiffeners. The maximum shear is equal t maximum reaction which in this case is obviously equal to the left rea Taking the center of moments at the right reaction, the equation of momen

$$R_1 \times 24.83' = (17 \text{ tons} \times 23.33') + (21.15 \text{ tons} \times 15.91') + (21.15 \text{ tons} \times 8.$$
$$+ (15.6 \text{ tons} \times 1.33')$$

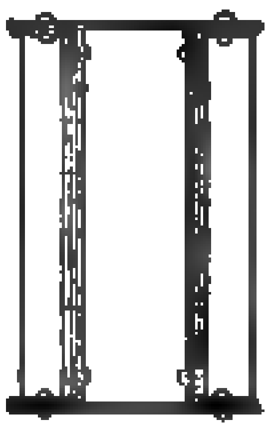
whence $24.83 \times R_1 = 935.3215$ ft-tons and $R_1 = 37.669$ tons, or 75 338 lb. that the loads have been changed from pounds to tons, for convenience in ing the calculations. As this box girder has two webs, the maximum sh each web will be 37 669 lb. The thinnest web permissible is ⅜ in thick. Table II, page 703, the resistance of a ⅜ by 30-in web-plate to sheari 112 500 lb, so that the webs are amply safe in resisting vertical shear. Table III, page 705, the safe resistance to buckling, deducting for two rivets, is 33 830 lb. As this is less than the maximum shear, stiffener be used, placed 2 ft 4 in from each support, with five between them, m the spacing about 3 ft 4 in on centers. Two others will be placed over support. 4 by 4 by ⅜-in angles will be sufficient for the stiffeners.

Note. If the loads were really concentrated at the points P_1, P_2, et from columns or girders, it would be necessary to place stiffeners at eac of these points and two in each of the intermediate spaces, but as the loads are partly distributed it will be better to space them as first planned

Fifth Step. The Number and Pitch of the Rivets. The rivets in the legs and flange-legs of the angles are in single shear. From Table III, 419, the shearing value of a ¾-in rivet in single shear at 10 000 lb per s 4 420 lb, and the bearing value in a ⅜-in plate at 18 000 lb per sq in is 5 Hence the shearing value will govern. The number of rivets required de upon the flange-stress, which is equal to the maximum bending m divided by the depth of the girder. (See Formula (1), page 683.) The b moment at P_1, found by moments or graphically by scaling off the or aa, Fig 17, is 56.5 ft-tons.* This, divided by the depth 2.5 ft, gives 22.

* This may be found, also, by taking P_1 as the center of moments and mult $R_1 = 37.669$ tons by the lever-arm 1½ ft. The result is 56.5 ft-tons. The b moments at the other loads may be determined by taking, in each case, the al sum of the moments of the external vertical forces on either side of each point con

..s 300 lb, for the flange-stress, or 22 600 lb for each web. The number of ..ts, therefore (Formula (5), page 687) is 22 600/4 420 = 6. The distance .. P_1 to the left reaction is 18 in, which makes the spacing 3 in. The ..e-stress at P_2 is 209.88 ft-tons/2.5 ft = 83.95 tons, or 167 900 lb, and one-.. of this is 83 950 lb. The number of rivets therefore is 83 950/4 420 = 19. .. 6 of these are required between P_1 and the left reaction, leaving 13 to go ..een P_1 and P_2, a distance of 89 in, making the pitch about 6.9 in. As .. exceeds the maximum allowable pitch, the rivets will be spaced 6 in on ..ters between P_1 and P_2, and between P_2 and P_3. The spacing on the right-..d end of the girder will be made the same as that on the left. Some details ..e girder are shown in Fig. 18.



..irder. Example 2

.. **Details and Bill of Quantities for the Girder.** The loads, dimensions, size, ..er of pieces, etc., for the girder are given in the following summary:

..ds: 34 000 lb, 1 ft 6 in from left support. Span: 24 ft 10 in
42 300 lb, 8 ft 11 in from left support Depth: 30 in
42 300 lb, 8 ft 7 in from right support
31 200 lb, 1 ft 4 in from right support
Both flanges: Four angles, 5 by 3½ by 7/16 in, 27 ft 6 in long
One plate, 20 by ¾ in, 27 ft 6 in long
..wo webs: ⅜ by 30 in, 27 ft 6 in long
Twenty-two stiffeners: 4 by 4 by ⅜ in, 29¼ in long
Twenty-two filler-plates: 4 by 7/16 in, 23 in long
..ivets: ¾ in in diameter

..ple 3. What are the dimensions of a box girder, 40 ft in span, required to ..rt the following loads? 90 tons from a column, 8 ft from the left support; ..s from a column, 12 ft from the right support; and a masonry pier, 10 ft ..gth, beginning 10 ft from the left support and weighing 4 tons per running (See Fig. 19.)

.. **Step. The Determination of the Reactions, Shears and Bending Mo-..** To find either reaction, the center of moments is taken at the other ..on. The equation of moments for the left reaction is, therefore, taking the ..f moments at the right reaction,

$$40\,R_1 = (90 \text{ tons} \times 32 \text{ ft}) + (40 \text{ tons} \times 25 \text{ ft}^*) + (75 \text{ tons} \times 12 \text{ ft})$$

..hich

$$40\,R_1 = 4\,780 \text{ ft-tons and } R_1 = 119.5 \text{ tons}$$

..onsidering the moments of forces, distributed loads are treated as if they were ..trated at their centers of gravity.

In like manner, the equation of moments for the right reaction is

$$40\,R_2 = (75 \text{ tons} \times 28 \text{ ft}) + (40 \text{ tons} \times 15 \text{ ft}) + (90 \text{ tons} \times 8 \text{ ft})$$

from which

$$40\,R_2 = 3\,420 \text{ ft-tons, and } R_2 = 85.5 \text{ tons}$$

The greatest vertical shear V_1 is equal to the greater reaction, whic[h is] 119.5 tons. The shear-diagram (Fig. 19) may be constructed by laying o[ff]

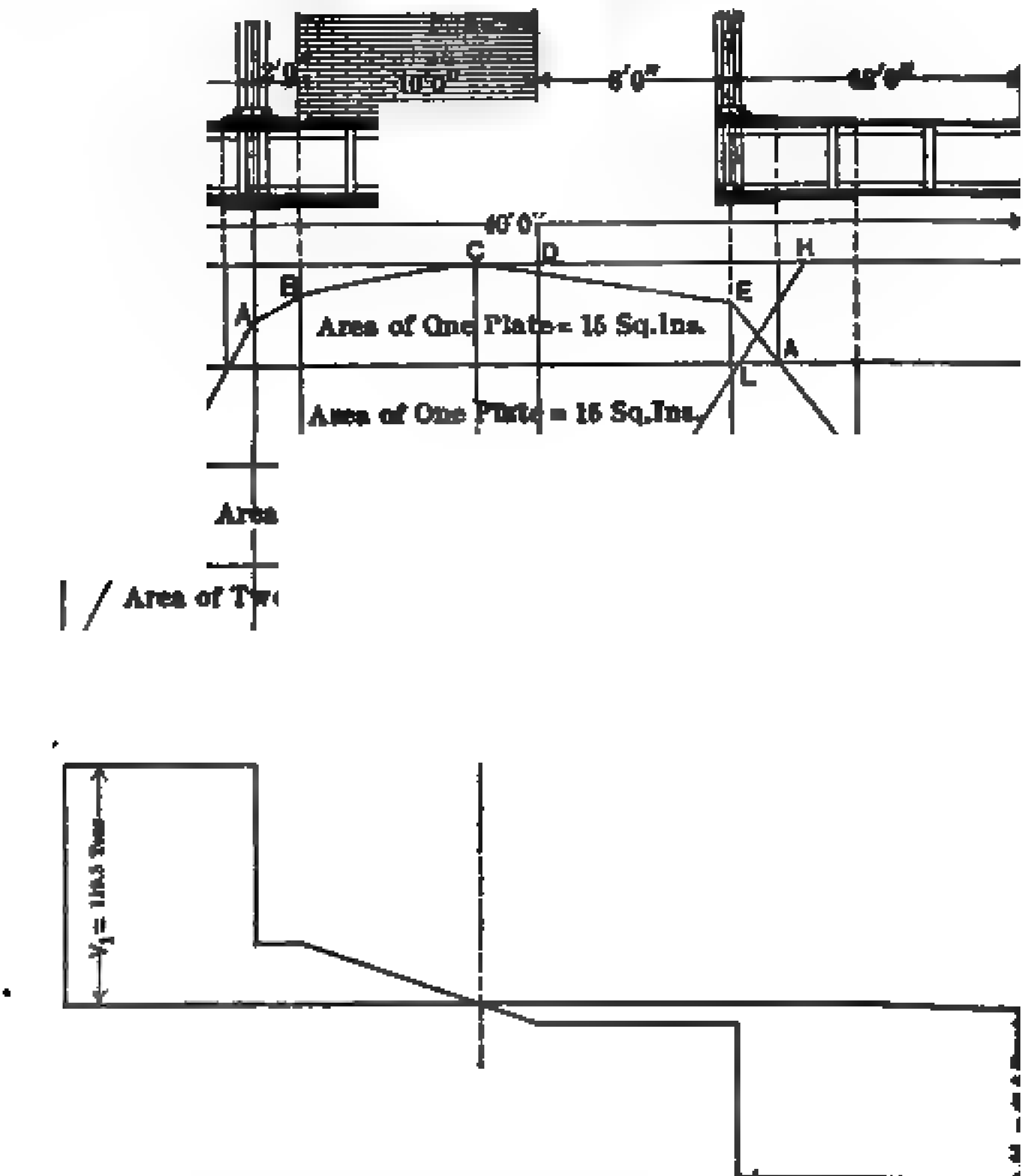


Fig. 19. Elevation of Box Girder and Diagrams for Bending Moments and V[ertical] Shears. Example 3

any convenient scale an ordinate equal in length to 119.5 tons. Immed[iately] at the right of point 1, under the left column, the shear is equal to 119.5 – [90 =] 29.5 tons. It is the same at point 2, the left end of the wall. At point [3, the] right end of the wall, the shear is 119.5 – 90 – 40 = – 10.5 tons, showing t[hat the] shear passes through zero somewhere between 2 and 3, which is the p[oint of] maximum bending moment. This point, X, is by scaling the shear-dia[gram] 17.4 ft from R_1. It is over the point of intersection of the slanting l[ine]

e shear-diagram, with the horizontal line of reference. This slanting line drawn from the top of the shear-ordinate for point 2 to the bottom of the ar-ordinate for point 3. Just at the right of point 4, the shear is 119.5 − − 40 − 75 = −85.5 tons, the same as the right reaction. The point X, of no ar and maximum bending moment, may be found, also, as follows: At the t of point 2 the shear is 29.5 tons. One foot to the right of 2 it is 29.5 s − 4 tons = 25.5 tons. Two feet to the right of 2 it is 29.5 − 8 tons = .5 tons, etc. Therefore, since the shear decreases at the rate of 4 tons per ot, it will be zero at 29.5/4 or 7.4 ft at the right of 2, or 17.4 ft from R_1.

The maximum bending moment is at X, the point of no shear. The equation moments, considering the forces to the left of X, is

$$M_{max} = (119.5 \text{ tons} \times 17.4 \text{ ft}) - (90 \text{ tons} \times 9.4 \text{ ft}) - (4 \text{ tons} \times 7.4 \text{ ft} \times 7.4 \text{ ft}/2)$$

ft/2 is the distance from X to the center of gravity of the wall-load to the t of X, and is the lever-arm of that load, considered as a vertical downward ce concentrated in a single line of action. Hence

$$M_{max} = 2\,079.3 - 846 - 109.5 = 1\,123.8 \text{ ft-tons}$$

The bending-moment diagram may be constructed by laying off at X, at any venient scale, an ordinate XC equal to 1 123.8 ft-tons in length. It is neces- y to find the bending moment at other points, since the bending-moment gram cannot be plotted, as in the previous examples, because the uniform d is not distributed over the entire girder. The other critical points are 1, 3 and 4.

$$M_1 = (119.5 \text{ tons} \times 8 \text{ ft}) = 956 \text{ ft-tons}$$

$$M_2 = (119.5 \text{ tons} \times 10 \text{ ft}) - (90 \text{ tons} \times 2 \text{ ft}) = 1\,015 \text{ ft-tons}$$

$$M_3 = (119.5 \text{ tons} \times 20 \text{ ft}) - (90 \text{ tons} \times 12 \text{ ft}) - (40 \text{ tons} \times 5 \text{ ft}) = 1\,110 \text{ ft-tons}$$

$$M_4 = (119.5 \text{ tons} \times 28 \text{ ft}) - (90 \text{ tons} \times 20 \text{ ft}) - (40 \text{ tons} \times 13 \text{ ft}) = 1\,026 \text{ ft-tons}$$

y laying off ordinates at these points equal by scale to the respective bending ments; drawing straight lines from R to A, the extremity of the ordinate ugh 1, and from A to B; drawing curved lines from B through the points nd D; and connecting D and E and E and R_2 by straight lines; the bending- ment diagram $R_1ABCDER_2$ may be constructed.

econd Step. The Webs. As stated on page 683, it is considered safe by y engineers to include one-sixth of the web-area in the flange-area, and will be done in this example. The web, therefore, must be designed first. there is nothing to limit the depth of the girder, it will be made 3 ft deep, t one-twelfth the span. The greatest vertical shear is equal to the greater ft reaction, 119.5 tons. Since the girder carries a brick wall, it must be of ox type, and hence the vertical shear on each web is 59.75 tons. A ½ by web will be tried first. Its area is 18 sq in, from which must be deducted ss in area due to the rivet-holes for the rivets through the stiffeners. The will be placed the maximum distance on centers, making six in each er. Because of the concentrated loads near the reactions more rivets will quired, and in order to avoid a close spacing, ⅞-in rivets will be used. Table I, page 702, the sectional area to be deducted for a ⅞-in rivet in a plate is 0.50 sq in; hence the net area of the web is 18 − (6 × 0.50) = 15 and its shearing resistance, at 10 000 lb or 5 tons per sq in (Table II, page is 15 × 5 tons = 75 tons, which is 15.25 tons in excess of the 59.75 tons ed.

d Step. The Flange-Area. From Formula (1), page 683,

$$\text{the flange area, } A = M_{max}/dS = \frac{1\,123.8}{3 \times 7} = 53.5 \text{ sq in}$$

As the girder has no lateral support, the flange-width should be not less than (twentieth the span, which will make it 2 ft.

The upper flange may be proportioned as follows:

One-sixth of net section-area of two webs =	5.00 sq in
Two 5 by 5 by 9/16-in angles,* with section-area =	10.62 sq in
Three 9/16 by 24-in plates, with section-area of 13.50 sq in each =	40.50 sq in
Total section-area of upper flange =	56.12 sq in

To proportion the lower flange, allowance must be made for the loss in area due to two rivet-holes.

From Table I, page 702, the area of two 7/8-in rivet-holes in a 9/16-in plate (thickness of angles) =	1.12 sq in
Area† of two rivet-holes in three 5/8-in flange-plates =	3.75 sq in
Total rivet-area =	4.87 sq in

Hence the gross section-area of the lower flange must be $53.5 + 4.87 = 58$ sq in.

This may be made up of

One-sixth of net section-area of two webs =	5.0 sq in
Two 5 by 5 by 9/16-in angles, with section-area =	10.62 sq in
Three 5/8 by 24-in plates, with section-area of 15 sq in each =	45.00 sq in
Total section-area of lower flange‡ =	60.62 sq in

The length of the flange-plates is determined from the bending-mo diagram. Draw a horizontal line through *C* (Fig. 19) and at any point, lay off to any convenient scale and angle, a line $3\,H = 60.62$ units in le with its upper extremity on the horizontal line *FG* drawn through *C*. D this line into five parts: $3\,I$, containing 5 units for the web-area; *IJ*, 10.62 for the angles; and *JK*, *KL* and *LH* of 15 units each, for the three plates. horizontal lines through the points *I*, *J*, *K* and *L* as shown. The horiz intercepts of these horizontals in the bending-moment diagram will giv theoretical lengths of the flange-plates. For practical considerations, the plate is always carried the full length of the girder and the other plates a tended beyond the intersection-points on either side, a distance sufficie take enough rivets to transmit at least one-third of the resistance of the The resistance *AS*, of the outer plate is 15 sq in × 14 000 lb per sq in = 210 o One-third of this, or 70 000 lb, must be resisted by rivets placed beyon points *AA*. From Table III, page 419, at 10 000 lb per sq in, the sh value of a 7/8-in steel rivet in single shear is 6 010 lb and in a 5/8-in plate its be value at 18 000 lb per sq in is 9 820 lb. Hence the number of rivets requi 70 000/6 010 = 12, or 6 on each side. With a 2-in pitch this would length plate 12 in at each end. The outside plates in this particular girder wo extended far enough to pass beyond the base of the column on the left s the girder.

* Angles with equal legs are selected because the same number of rivets will quired in both legs, as they are all in single shear, and large angles are selected b the rivets will have to be staggered, owing to the concentrated loads being placed the ends of the girder.

† Since 9/16-in plates are selected for the upper flange, it is reasonable to suppo 5/8-in plates will be necessary for the lower flange.

‡ Both flange-areas are made slightly in excess of the requirements, because example one-sixth of the web-area is included.

Fourth Step. The Stiffeners. From Table III, page 705, the safe buckling value of a ½ by 36-in plate with two ⅞-in rivets is 62 320 lb, and as this is much less than the shearing value, stiffeners must be used. The stiffeners under the concentrated loads may be considered as short struts in direct compression. Assuming that 4 by 4 by ½-in angles are used for the stiffeners, the safe load from Table XV, page 502, is over 20 tons. The greatest concentrated load is 90 tons, and hence four stiffeners will be placed under each column. Four more will be placed at each bearing, as shown in Fig. 19, four on each side, between the columns, about 4 ft on centers; and two on each side, between the columns and the bearings, making 15 on each side, or 30 in all.

Fifth Step. The Number and Pitch of the Rivets. In a box girder, the rivets are in single shear. The shearing value of a ⅞-in rivet at 10 000 lb per sq in is, from Table III, page 419, 6 010 lb, and its bearing value at 18 000 lb per sq in, in a 7/16-in plate, the thinnest outside plate, is 6 880 lb; hence the shearing value will govern.

The number of rivets depends upon the horizontal flange-stress, which is equal to the maximum bending moment divided by the depth of the girder (Formula (1), page 683). M at 1 = 956 ft-tons, and the horizontal flange-stress = 956/3 = 319 tons, or 638 000 lb. From Formula (5), page 687, the number of rivets required = 638 000/6 010 = 106, or 53 on each side. These are to be placed in a distance of 8 ft, or 96 in, which makes the pitch about 1.8 in. As this is less than the minimum pitch, 2⅝ in, or three diameters, the rivets will have to be staggered. Hence the justification for selecting large angles with equal legs for this paricular girder. At X the horizontal flange-stress = 1123.8/3 = 374.6 tons, or 749 200 lb, and the number of rivets is 749 200/6 010 = 124, or 62 on each side; 53 of these, however, are required between R_1 and 1, leaving 9 to be placed between 1 and X, a distance of about 9 ft. As the resulting pitch will exceed the maximum pitch, they will be placed 6 in on centers between 1 and X. At 4 the horizontal flange-stress = 1 026/3 = 342 tons, or 684 000 lb. The number of rivets is 684 000/6 010 = 112, or 56 on each side, to be spaced in a distance of 12 ft, or 144 in, making the spacing 2.5 in. Between 4 and X the maximum pitch will be determined as before.

Sixth Step. The Weight of the Girder. So far, no account has been taken of the weight of the girder. The practice is to neglect this weight when the maximum bending moment due to it alone is less than 10% of the maximum bending moment due to the loads. From Formula (7), page 688, the weight of the girder = 205 × 40/700 = 12 tons. From Case V, page 326, the maximum bending moment due to it = 12 × 40/8 = 60 ft-tons. As this is much less than 10% of 1123.8 ft-tons, the maximum bending moment due to the loads, it may be neglected. Had it been otherwise, the weight would have to be considered as an additional uniformly distributed load over the entire girder and a new bending-moment diagram drawn.

Further Data on Riveted Girders. By applying the principles illustrated in the preceding examples it is possible to compute the necessary dimensions and details for riveted girders under any conditions of loading. If further examples are desired, the reader is referred to "Compound Riveted Girders," by William Birkmire, in which different examples of loading are fully worked and explained, and also to other recent treatises on this subject.

Detail Drawings and Stress-Diagrams of one of the earlier heavy plate girders used in building-construction are published in the Engineering Record of Dec. 28, 1895. This girder is one of six plate girders used in the construction of Tremont Temple, Boston, Mass., Blackall & Newton, architects. The girder is 75 ft long between centers of columns, 6 ft 1 in deep, with flanges 28 in

wide, and is calculated to support distributed and concentrated loads ag gating 497.5 tons. The single web-plate is 64¾ in deep, and ⅞ in thick at ends; the flanges are 4½ in thick at the middle of the girder; and the fla angles are 6 by 8 by 1 in. Since that time there have been erected for man the large buildings a number of riveted girders of very great size and stren and details of their construction may be found in the engineering and archi tural periodicals.

6. Tables Used in the Design of Plate and Box Girders

Tables I, II, III and IV contain data usually required for the design of p and box girders to satisfy all but the most unusual conditions.

Table I.*† Sectional Area in Square Inches to be Deducted from Plates Angles for Rivet-Holes

Taken ⅛ inch in excess of diameter of rivet‡

Thickness of plate, in	Number of rivets, 1 in diameter				Number of rivets, ⅞ in diameter			
	1	2	3	4	1	2	3	4
1	1.12	2.25	3.37	4.50	1.00	2.00	3.00	4.
15/16	1.05	2.10	3.16	4.21	0.94	1.87	2.81	3.
⅞	0.98	1.97	2.95	3.93	0.87	1.75	2.62	3.
13/16	0.91	1.83	2.74	3.65	0.81	1.62	2.44	3.
¾	0.84	1.69	2.53	3.37	0.75	1.50	2.25	3.
11/16	0.77	1.55	2.32	3.09	0.69	1.37	2.06	2.
⅝	0.70	1.41	2.11	2.81	0.62	1.25	1.87	2.
9/16	0.63	1.26	1.90	2.53	0.56	1.12	1.69	2.
½	0.56	1.11	1.69	2.25	0.50	1.00	1.50	2.
7/16	0.49	0.98	1.47	1.97	0.44	0.87	1.31	1.
⅜	0.42	0.84	1.26	1.69	0.37	0.75	1.12	1.

Thickness of plate, in	Number of rivets, ¾ in diameter				Number of rivets, ⅝ in diameter			
	1	2	3	4	1	2	3	4
1	0.87	1.75	2.62	3.50	0.75	1.50	2.25	3.
15/16	0.82	1.64	2.46	3.28	0.70	1.40	2.11	2.
⅞	0.77	1.53	2.30	3.06	0.65	1.31	1.96	2.
13/16	0.71	1.42	2.13	2.84	0.61	1.22	1.83	2.
¾	0.66	1.31	1.96	2.62	0.56	1.12	1.69	2.
11/16	0.60	1.20	1.80	2.40	0.51	1.03	1.54	2.
⅝	0.55	1.09	1.64	2.19	0.47	0.94	1.41	1.
9/16	0.49	0.98	1.48	1.96	0.42	0.84	1.26	1.
½	0.43	0.87	1.31	1.75	0.37	0.75	1.12	1.
7/16	0.38	0.76	1.15	1.53	0.33	0.66	0.98	1.
⅜	0.32	0.65	0.98	1.31	0.28	0.56	0.84	1.
5/16	0.27	0.55	0.82	1.09	0.23	0.47	0.70	0.
¼	0.22	0.44	0.66	0.87	0.18	0.37	0.56	0.

* For explanation of tables, see Subdivision 4, page 688.
† This table is taken from "Compound Riveted Girders," by W. H. Birkmire.
‡ See paragraph, Punching Rivet-Holes, page 414, and Table XI, page 400.

Table II.* Safe Shearing Value of Web-Plates in Pounds

Mild steel. Gross area. Safe unit stress, 10 000 lb per sq in

...pth, in	Thickness in inches						
	3/8	7/16	1/2	9/16	5/8	3/4	7/8
28	105 000	122 500	140 000	157 500	175 000	210 000	245 000
30	112 500	131 300	150 000	168 800	187 500	225 000	262 500
32	120 000	140 000	160 000	180 000	200 000	240 000	280 000
36	135 000	157 500	180 000	202 500	225 000	270 000	315 000
40	150 000	175 000	200 000	225 000	250 000	300 000	350 000
42	157 500	183 800	210 000	236 300	262 500	315 000	367 500
46	172 500	201 300	230 000	258 800	287 500	345 000	402 500
48	180 000	210 000	240 000	270 000	300 000	360 000	420 000
	Deductions in pounds for one 3/4-in rivet†						
	3 200	3 800	4 300	4 900	5 500	6 600	7 700
	Deductions in pounds for one 7/8-in rivet†						
	3 700	4 400	5 000	5 600	6 200	7 500	8 700

* For explanation of tables, see Subdivision 4, page 688.

† The area of the hole is taken 1/8 in in excess of the diameter of the rivet to allow for ...ry of the metal sustained by punching.

...ample 4. What is the safe shearing value of a 36 by 3/8-in web-plate with ...en 3/4-in rivets in the stiffeners?

...lution. The gross shearing value = 135 000 lb
The deduction for seven rivets = 7 × 3 200 = 22 400 lb
The safe shearing value = 112 600 lb

...use this table for any other unit stress, divide the shearing value by 10 000 ... multiply by the given unit stress. For example, what is the safe shearing-...ue of a 40 by 5/8-in web-plate at 12 000 lb per sq in? (250 000/10) × 12 = ...000 lb.

...ables of Riveted Steel Plate Girders.‡ It is not practicable to give ...ES OF SAFE LOADS for riveted steel plate girders because of the great variety ...mbinations of plates and angles that can be selected for any given condition ...ding. Moreover, any variation in the loading would make the tables use-... In place of the safe loads, therefore, the PROPERTIES OR ELEMENTS OF ...ED STEEL PLATE GIRDERS are given in Table IV, pages 706 to 716, which ... aid in determining the size of the girder and the approximate thickness of ...plates and angles for any special case. To determine the dimensions and ... details of a girder suitable to carry any specified loading, determine the ...UM END-REACTION in pounds and the MAXIMUM BENDING MOMENT in inch-...ds. Select from Table IV the different parts for a girder of the required ..., a THICKNESS OF WEB as determined by the maximum end-reaction and ...table SECTION-MODULUS as determined by dividing the maximum bending

‡ For tables of riveted single-beam girders and double-beam girders, see Tables XIV ...XV, pages 605 to 611.

moment by the PERMISSIBLE UNIT STRESS FOR FLEXURE in pounds per sq inch. The SPACING OF THE RIVETS, the number and position of the STIFFEN the LENGTH OF THE FLANGE-PLATES, if more than one are needed, and the IN FLANGE-AREA and WEB-AREA due to the punching of the RIVET-HOLES, be determined in each case by the rules already given. The weights of rivets and stiffeners are not included.

As an illustration of the use of these elements or properties, in Exampl the total load on the girder is 107 000 lb, making each end-reaction 53 50 The maximum bending moment is 334.8 ft-tons, or 8 035 000 in-lb. The sec modulus $I/c = M/S = 8\,035\,000/14\,000 = 574$. The depth of the girde limited to 36 in. Looking up the properties of 36-in girders in Table IV, 709, it is seen that a ⅜-in web is more than sufficient to resist the reaction. The nearest section-modulus to 574 is 567.2, that of a girder posed of a 36 by 7/16-in web, 5 by 3½ by ½-in angles, and 12 by ½-in fl plates. In working out the problem in detail it was found that the g required 5 by 3½ by 7/16-in angles and two 12 by ½-in flange-plates to pensate for the loss of area due to the punching of the rivet-holes. (See 219 and 710.)

Table IV is based upon an extreme fiber-stress for flexure of 16 000 lb per and gross sections are used in determining the values given. The attenti readers is called to the two methods of plate and box-girder design: (1) th using the plate-girder formula (page 683), and (2) the one using the sec modulus (pages 703 and 704, and 706 to 716). It is customary, also, to into account the tendency of the compression-flange of the girder, if long bet lateral bracings, to buckle or fail as a column; and the permissible reduced fi stress is determined by column-formulas.

Table III.* Safe Buckling Values of Web-Plates

SAFE UNIT BUCKLING VALUE IN POUNDS PER SQUARE INCH

$$\text{Calculated by formula† } S_b = \frac{10\,000}{1 + \dfrac{d^2}{3\,000\,t^2}}$$

S_b = safe buckling resistance in pounds per square inch; d = depth of web in the clea[r] [be]tween flange-plates in inches; t = thickness of web in inches

Depth, in	Thickness in inches						
	3/8	7/16	1/2	9/16	5/8	3/4	7/8
28	3 498	4 228	4 890	5 476	5 932	....	....
30	3 192	3 896	4 546	5 133	5 656	6 522	....
32	2 889	3 624	4 228	4 787	5 339	6 226	6 920
36	2 456	3 069	3 666	4 229	4 748	5 656	6 392
40	2 087	2 696	3 191	3 724	4 228	5 133	5 882
42	1 930	2 455	2 983	3 498	3 992	4 889	5 649
48	1 548	1 994	2 543	2 918	3 371	4 228	4 992

TOTAL SAFE RESISTANCE IN POUNDS FOR PLATES WITH TWO 3/4-IN RIVETS

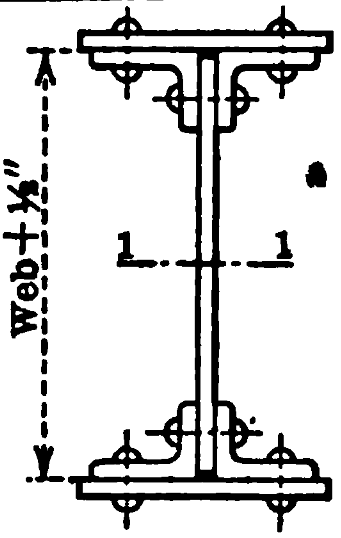
Depth, in	Thickness in inches						
	3/8	7/16	1/2	9/16	5/8	3/4	7/8
28	34 450	48 580	64 200	80 880	97 340	....	....
30	33 830	48 150	64 230	81 560	99 880	138 200	....
36	31 560	46 000	62 800	81 500	101 750	145 300	191 570
42	29 140	43 230	60 040	79 190	100 440	147 600	198 960
48	26 860	40 360	58 820	75 920	97 450	146 670	202 000

TOTAL SAFE RESISTANCE IN POUNDS FOR PLATES WITH TWO 7/8-IN RIVETS

Depth, in	Thickness in inches						
	3/8	7/16	1/2	9/16	5/8	3/4	7/8
28	34 100	48 110	63 570	80 100	96 390	....	....
30	33 510	47 720	63 640	80 840	98 980	136 960	....
36	31 310	45 660	62 320	80 900	100 690	144 230	190 170
42	28 950	42 960	59 660	78 700	99 800	146 690	197 710
48	26 700	40 140	58 490	75 520	96 910	145 860	200 930

* For explanation of tables, see Subdivision 4, page 688.
† See in Chapter XV the paragraphs and foot-notes, pages 568 and 569, relating tc web-buckling of I-beams. The formula for the above table is the formula that was [used] in the Passaic Steel Company's Manual, and as the values computed by it vary but [little] from those deduced by the Cambria formula, Table III is retained as it is.
See, also, page 686, paragraph relating to Safe Resistance of Web to Buckling.

Table IV.* Elements of Riveted Plate Girders



To determine the details of construction of a girder su[itable] able to carry any specified loading, determine the ma[ximum] mum end-reactions in pounds and the maximum bendi[ng] moment in inch-pounds

Select from the table a girder having the desired dep[th] a thickness of web as determined by the maximum en[d] reaction and a suitable section-modulus, determined [by] dividing the maximum bending moment by the permis[si]ble unit bending fiber-stress in pounds per square inc[h]

For limiting conditions, see the pages 702 to 705 a[nd] the first three subdivisions of this chapter

Weights given do not include stiffeners, rivet-heads, [or] other details

Section-modulus, axis, 1–1, in³	Sizes: Web-plate, in	Sizes: Flange-angles, in	Sizes: Flange-plates, in	Weight per foot: Web-plate and flange-angles, lb	Weight per foot: Flange-plates, lb	Maxim[um] end-reactio[ns] thousa[nds] of poun[ds]
242.0	24×⅜	5×3½×⅝		97.8		60.8
270.9		5×3½×⅜	12×⅜	72.2	30.6	60.8
306.1		5×3½×⅜	12×½	72.2	40.8	60.[illegible]
343.6		5×3½×½	12×½	85.0	40.8	60.[illegible]
378.5		5×3½×½	12×⅝	85.0	51.0	60.[illegible]
414.1		5×3½×⅝	12×⅝	97.8	51.0	60.[illegible]
151.5	26×5/16	4×3 ×⅜		61.6		56.[illegible]
176.8		5×3½×⅜		69.2		56.[illegible]
186.6		4×3 ×½		72.0		56.[illegible]
201.2		6×4 ×⅜		76.8		56.[illegible]
219.6		5×3½×½		82.0		56.[illegible]
252.0		6×4 ×½		92.4		56.[illegible]
260.7		5×3½×⅝		94.8		56.[illegible]
341.5	26×⅜	6×4 ×⅜	14×⅜	82.4	35.7	67.[illegible]
354.4		6×4 ×¾		127.6		67.[illegible]
377.4		5×3½×½	12×½	87.6	40.8	67.[illegible]
386.1		6×4 ×⅜	14×½	82.4	47.6	67.[illegible]
415.2		5×3½×½	12×⅝	87.6	51.0	67.[illegible]
435.1		6×4 ×½	14×½	98.0	47.6	67.[illegible]
454.5		5×3½×⅝	12×⅝	100.4	51.0	67.[illegible]
479.3		6×4 ×½	14×⅝	98.0	59.5	67.[illegible]
526.1		6×4 ×⅝	14×⅝	113.2	59.5	67.[illegible]
569.9		6×4 ×⅝	14×¾	113.2	71.4	67.[illegible]
613.9		6×4 ×¾	14×¾	127.6	71.4	67.[illegible]
200.4	26×7/16	4×3 ×½		83.1		78
233.4		4×3 ×⅝		93.1		78
233.5		5×3½×½		93.1		78
265.8		6×4 ×½		103.5		78
274.5		5×3½×⅝		105.9		78
314.8		6×4 ×⅝		118.7		78

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa. Som[e addi]tional values are given in the Pocket Companion.

Table IV *† (Continued). Elements of Riveted Plate Girde

Section-modulus, axis 1–1, in³	Sizes			Weight per foot	
	Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb
361.3	26×7/16	6×4 ×¾		133.1	
384.0		5×3½×½	12×½	93.1	40.8
421.8		5×3½×½	12×⅝	93.1	51.0
441.7		6×4 ×½	14×½	103.5	47.6
461.1		5×3½×⅝	12×⅝	105.9	51.0
485.9		6×4 ×½	14×⅝	103.5	59.5
532.7		6×4 ×⅝	14×⅝	118.7	59.5
576.5		6×4 ×⅝	14×¾	118.7	71.4
620.5		6×4 ×¾	14×¾	133.1	71.4
185.6	27×5/16	5×3½×⅜		70.3	
211.0		6×4 ×⅜		77.9	
230.3		5×3½×½		83.1	
264.1		6×4 ×½		93.5	
273.2		5×3½×⅝		95.9	
304.5		5×3½×⅜	12×⅜	70.3	30.6
315.3		6×4 ×⅝		108.7	
344.2		5×3½×⅜	12×½	70.3	40.8
337.7	28×⅜	6×4 ×⅝		115.7	
366.7		5×3½×⅜	12×½	77.3	40.8
372.8		6×4 ×⅜	14×⅜	84.9	35.7
388.5		6×4 ×¾		130.1	
411.7		5×3½×½	12×½	90.1	40.8
420.8		6×4 ×⅜	14×½	84.9	47.6
437.0		6×4 ×⅞		144.5	
452.5		5×3½×½	12×⅝	90.1	51.0
474.3		6×4 ×½	14×½	100.5	47.6
495.3		5×3½×⅝	12×⅝	102.9	51.0
521.9		6×4 ×½	14×⅝	100.5	59.5
573.1		6×4 ×⅝	14×⅝	115.7	59.5
620.4		6×4 ×⅝	14×¾	115.7	71.4
668.6		6×4 ×¾	14×¾	130.1	71.4
257.1	28×7/16	5×3½×½		96.1	
292.4		6×4 ×½		106.5	
301.8		5×3½×⅝		108.9	
345.8		6×4 ×⅝		121.7	
396.5		6×4 ×¾		136.1	
419.5		5×3½×½	12×½	96.1	40.8
445.1		6×4 ×⅞		150.5	
460.2		5×3½×½	12×⅝	96.1	51.0
482.0		6×4 ×½	14×½	106.5	47.6
503.0		5×3½×⅝	12×⅝	108.9	51.0
529.6		6×4 ×½	14×⅝	106.5	59.5
580.8		6×4 ×⅝	14×⅝	121.7	59.5

* From Pocket Companion, Carnegie Steel Company, Pittsburgh,

† For explanation of table, see page 706.

Table IV *† (Continued). Elements of Riveted Plate Girders

Section-modulus, axis 1-1, in³	Sizes			Weight per foot		Maxim end-reaction thousa of pound
	Web-plate, in	Flange-angles, in	*Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb	
628.0	28×7/16	6×4 ×5/8	14×3/4	121.7	71.4	78.8
676.2		6×4 ×3/4	14×3/4	136.1	71.4	78.8
221.8		5×3½×3/8		79.9		74.3
250.5		6×4 ×3/8		87.5		74.3
272.1		5×3½×½		92.7		74.3
310.3		6×4 ×½		103.1		74.3
320.5		5×3½×5/8		105.5		74.3
353.8		5×3½×3/8	12×3/8	79.9	30.6	74.3
366.2		5×3½×3/4		117.5		74.3
368.1		6×4 ×5/8		118.3		74.3
397.8		5×3½×3/8	12×½	79.9	40.8	74.3
404.7		6×4 ×3/8	14×3/8	87.5	35.7	74.3
423.1	30×3/8	6×4 ×3/4		132.7		74.3
446.6		5×3½×½	12×½	92.7	40.8	74.3
456.1		6×4 ×3/8	14×½	87.5	47.6	74.3
475.8		6×4 ×7/8		147.1		74.3
490.3		5×3½×½	12×5/8	92.7	51.0	74.3
514.0		6×4 ×½	14×½	103.1	47.6	74.3
536.7		5×3½×5/8	12×5/8	105.5	51.0	74.3
565.1		6×4 ×½	14×5/8	103.1	59.5	74.3
620.6		6×4 ×5/8	14×5/8	118.3	59.5	74.3
671.3		6×4 ×5/8	14×3/4	118.3	71.4	74.3
723.8		6×4 ×3/4	14×3/4	132.7	71.4	74.3
281.4		5×3½×½		99.0		86.6
319.5		6×4 ×½		109.4		86.6
329.7		5×3½×5/8		111.8		86.6
375.5		5×3½×3/4		123.8		86.6
377.3		6×4 ×5/8		124.6		86.6
432.3		6×4 ×3/4		139.0		86.6
455.5		5×3½×½	12×½	99.0	40.8	86.6
485.0	30×7/16	6×4 ×7/8		153.4		86.6
499.2		5×3½×½	12×5/8	99.0	51.0	86.6
523.0		6×4 ×½	14×½	109.4	47.6	86.6
545.6		5×3½×5/8	12×5/8	111.8	51.0	86.6
574.0		6×4 ×½	14×5/8	109.4	59.5	86.6
629.5		6×4 ×5/8	14×5/8	124.6	59.5	86.6
680.1		6×4 ×5/8	14×3/4	124.6	71.4	86.6
732.6		6×4 ×3/4	14×3/4	139.0	71.4	86.6
290.6		5×3½×½		105.4		99.0
328.8		6×4 ×½		115.8		99.0
338.9	30×½	5×3½×5/8		118.2		99.0
384.7		5×3½×3/4		130.2		99.0
386.5		6×4 ×5/8		131.0		99.0

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.
† For explanation of table, see page 706.

Table IV*† (Continued). Elements of Riveted Plate Girders

Section-modulus, axis 1-1, in^3	Sizes			Weight per foot		Maximum end-reaction in thousands of pounds
	Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb	
441.5	30×½	6×4 ×¾		145.4		99.0
464.4		5×3½×½	12×½	105.4	40.8	99.0
494.2		6×4 ×⅞		159.8		99.0
508.0		5×3½×½	12×⅝	105.4	51.0	99.0
531.9		6×4 ×½	14×½	115.8	47.6	99.0
554.5		5×3½×⅝	12×⅝	118.2	51.0	99.0
582.8		6×4 ×½	14×⅝	115.8	59.5	99.0
638.3		6×4 ×⅝	14×⅝	131.0	59.5	99.0
688.9		6×4 ×⅝	14×¾	131.0	71.4	99.0
741.3		6×4 ×¾	14×¾	145.4	71.4	99.0
251.7	33×⅜	5×3½×⅜		83.7		81.0
283.7		6×4 ×⅜		91.3		81.0
307.7		5×3½×½		96.5		81.0
308.4		6×6 ×⅜		101.7		121.5
350.3		6×4 ×½		106.9		81.0
430.3	36×⅜	6×6 ×½		124.3		135.0
460.0		5×3½×¾		125.1		87.8
462.4		6×4 ×⅝		125.9		87.8
503.3		6×4 ×⅜	14×⅜	95.1	35.7	87.8
510.5		6×6 ×⅝		142.7		135.0
530.2		6×4 ×¾		140.3		87.8
531.6		6×6 ×⅜	14×⅜	105.5	35.7	135.0
554.3		5×3½×½	12×½	100.3	40.8	87.8
565.1		6×4 ×⅜	14×½	95.1	47.6	87.8
593.2		6×6 ×⅜	14×½	105.5	47.6	135.0
595.3		6×4 ×⅞		154.7		87.8
606.8		5×3½×½	12×⅝	100.3	51.0	87.8
636.5		6×4 ×½	14×½	110.7	47.6	87.8
654.9		6×6 ×⅜	14×⅝	105.5	59.5	135.0
664.2		5×3½×⅝	12×⅝	113.1	51.0	87.8
674.4		6×6 ×½	14×½	124.3	47.6	135.0
698.0		6×4 ×½	14×⅝	110.7	59.5	87.8
735.5		6×6 ×½	14×⅝	124.3	59.5	135.0
766.6		6×4 ×⅝	14×⅝	125.9	59.5	87.8
796.8		6×6 ×½	14×¾	124.3	71.4	135.0
813.1		6×6 ×⅝	14×⅝	142.7	59.5	135.0
827.6		6×4 ×⅝	14×¾	125.9	71.4	87.8
873.8		6×6 ×⅝	14×¾	142.7	71.4	135.0
892.8		6×4 ×¾	14×¾	140.3	71.4	87.8
357.7	36×$\frac{7}{16}$	5×3½×½		108.0		102.4
404.7		6×4 ×½		118.4		102.4
417.0		5×3½×¾		120.8		102.4
443.6		6×6 ×½		132.0		157.5

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

† For explanation of table, see page 706.

Table IV*† (Continued). Elements of Riveted Plate Girders

Section-modulus, axis 1-1, in³	Sizes: Web-plate, in	Sizes: Flange-angles, in	Sizes: Flange-plates, in	Weight per foot: Web-plate and flange-angles, lb	Weight per foot: Flange-plates, lb	Maximum end-reaction in thousands of pounds
473.3	36×7/16	5×3½×¾		132.8		102.4
475.7		6×4 ×⅝		133.6		102.4
523.8		6×6 ×⅝		150.4		157.5
543.5		6×4 ×¾		148.0		102.4
567.2		5×3½×½	12×½	108.0	40.8	102.4
608.6		6×4 ×⅞		162.4		102.4
619.7		5×3½×½	12×⅝	108.0	51.0	102.4
649.5		6×4 ×½	14×½	118.4	47.6	102.4
677.1		5×3½×⅝	12×⅝	120.8	51.0	102.4
687.3		6×6 ×½	14×½	132.0	47.6	157.5
710.8		6×4 ×½	14×⅝	118.4	59.5	102.4
748.4		6×6 ×½	14×⅝	132.0	59.5	157.5
779.5		6×4 ×⅝	14×⅝	133.6	59.5	102.4
809.5		6×6 ×½	14×¾	132.0	71.4	157.5
825.9		6×6 ×⅝	14×⅝	150.4	59.5	157.5
840.4		6×4 ×⅝	14×¾	133.6	71.4	102.4
886.6		6×6 ×⅝	14×¾	150.4	71.4	157.5
905.5		6×4 ×¾	14×¾	148.0	71.4	102.4
418.0	36×½	6×4 ×½		126.0		117.0
456.9		6×6 ×½		139.6		180.0
489.0		6×4 ×⅝		141.2		117.0
537.1		6×6 ×⅝		158.0		180.0
556.9		6×4 ×¾		155.6		117.0
614.5		6×6 ×¾		176.0		180.0
621.9		6×4 ×⅞		170.0		117.0
662.5		6×4 ×½	14×½	126.0	47.6	117.0
689.2		6×6 ×⅞		193.6		180.0
700.3		6×6 ×½	14×½	139.6	47.6	180.0
723.7		6×4 ×½	14×⅝	126.0	59.5	117.0
761.3		6×6 ×½	14×⅝	139.6	59.5	180.0
792.3		6×4 ×⅝	14×⅝	141.2	59.5	117.0
822.3		6×6 ×½	14×¾	139.6	71.4	180.0
838.8		6×6 ×⅝	14×⅝	158.0	59.5	180.0
853.2		6×4 ×⅝	14×¾	141.2	71.4	117.0
899.4		6×6 ×⅝	14×¾	158.0	71.4	180.0
918.3		6×4 ×¾	14×¾	155.6	71.4	117.0
973.7		6×6 ×¾	14×¾	176.0	71.4	180.0
1 039.4		6×4 ×¾	14×1	155.6	95.2	117.0
1 094.1		6×6 ×¾	14×1	176.0	95.2	180.0
1 101.1		6×4 ×⅞	14×1	170.0	95.2	117.0
1 164.9		6×6 ×⅞	14×1	193.6	95.2	180.0
444.7	36×⅝	6×4 ×½		141.3		146.3
483.5		6×6 ×½		154.9		225.0

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.
† For explanation of table, see page 706.

Table IV * † (Continued). Elements of Riveted Plate Girder

Section-modulus, axis 1-1, in³	Sizes			Weight per foot	
	Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb
515.7	36×5/8	6×4×5/8		156.5	
563.7		6×6×5/8		173.3	
583.5		6×4×3/4		170.9	
641.2		6×6×3/4		191.3	
648.5		6×4×7/8		185.3	
688.4		6×4×1/2	14×1/2	141.3	47.6
715.8		6×6×7/8		208.9	
726.2		6×6×1/2	14×1/2	154.9	47.6
749.4		6×4×1/2	14×5/8	141.3	59.5
787.0		6×6×1/2	14×5/8	154.9	59.5
818.1		6×4×5/8	14×5/8	156.5	59.5
847.9		6×6×1/2	14×3/4	154.9	71.4
864.6		6×6×5/8	14×5/8	173.3	59.5
878.8		6×4×5/8	14×3/4	156.5	71.4
924.9		6×6×5/8	14×3/4	173.3	71.4
943.9		6×4×3/4	14×3/4	170.9	71.4
999.3		6×6×3/4	14×3/4	191.3	71.4
1 045.9		6×6×5/8	14×1	173.3	95.2
1 064.7		6×4×3/4	14×1	170.9	95.2
1 119.3		6×6×3/4	14×1	191.3	95.2
1 126.3		6×4×7/8	14×1	185.3	95.2
1 190.1		6×6×7/8	14×1	208.9	95.2
390.2	42×3/8	6×4×3/8		102.8	
427.5		6×6×3/8		113.2	
477.2		6×4×1/2		118.4	
527.2		6×6×1/2		132.0	
561.4		6×4×5/8		133.6	
606.6		6×4×3/8	14×3/8	102.8	35.7
623.5		6×6×5/8		150.4	
638.3		6×4×3/8	16×3/8	102.8	40.8
642.1		6×4×3/4		148.0	
643.2		6×6×3/8	14×3/8	113.2	35.7
675.1		6×6×3/8	16×3/8	113.2	40.8
678.6		6×4×3/8	14×1/2	102.8	47.6
715.2		6×6×3/8	14×1/2	113.2	47.6
716.5		6×6×3/4		168.4	
719.5		6×4×7/8		162.4	
757.7		6×6×3/8	16×1/2	113.2	54.4
763.7		6×4×1/2	14×1/2	118.4	47.6
787.2		6×6×3/8	14×5/8	113.2	59.5
806.2		6×4×1/2	16×1/2	118.4	54.4
806.4		6×6×7/8		186.0	
812.7		6×6×1/2	14×1/2	132.0	47.6

* From Pocket Companion, Carnegie Steel Company Pittsburgh,
† For explanation of table, see page 706.

le IV*† (Continued). Elements of Riveted Plate Girders

Sizes			Weight per foot		Maximum end-reaction i thousand of pounds
Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb	
42×⅜	6×4×½	14×⅝	118.4	59.5	101.3
	6×6×½	16×½	132.0	54.4	157.5
	6×6×½	14×⅝	132.0	59.5	157.5
	6×4×⅝	14×⅝	133.6	59.5	101.3
	6×6×½	16×⅝	132.0	68.0	157.5
	6×6×½	14×¾	132.0	71.4	157.5
	6×4×⅝	16×⅝	133.6	68.0	101.3
	6×6×⅝	14×⅝	150.4	59.5	157.5
	6×4×⅝	14×¾	133.6	71.4	101.3
	6×6×⅝	16×⅝	150.4	68.0	157.5
	6×6×⅝	14×¾	150.4	71.4	157.5
	6×4×¾	14×¾	148.0	71.4	101.3
	6×6×⅝	16×¾	150.4	81.6	157.5
	6×4×¾	16×¾	148.0	81.6	101.3
	6×6×¾	14×¾	168.4	71.4	157.5
	6×6×⅝	16×⅞	150.4	95.2	157.5
	6×6×¾	16×¾	168.4	81.6	157.5
	6×6×¾	16×⅞	168.4	95.2	157.5
	6×4×⅞	16×⅞	162.4	95.2	101.3
	6×6×⅞	16×⅞	186.0	95.2	157.5
42×7/16	6×4×½		127.3		118.1
	6×6×½		140.9		183.8
	6×4×⅝		142.5		118.1
	6×6×⅝		159.3		183.8
	6×4×¾		156.9		118.1
	6×6×¾		177.3		183.8
	6×4×⅞		171.3		118.1
	6×4×½	14×½	127.3	47.6	118.1
	6×4×½	16×½	127.3	54.4	118.1
	6×6×⅞		194.9		183.8
	6×6×½	14×½	140.9	47.6	183.8
	6×4×½	14×⅝	127.3	59.5	118.1
	6×6×½	16×½	140.9	54.4	183.8
	6×6×½	14×⅝	140.9	59.5	183.8
	6×4×⅝	14×⅝	142.5	59.5	118.1
	6×6×½	16×⅝	140.9	68.0	183.8
	6×6×½	14×¾	140.9	71.4	183.8
	6×4×⅝	16×⅝	142.5	68.0	118.1
	6×6×⅝	14×⅝	159.3	59.5	183.8
	6×4×⅝	14×¾	142.5	71.4	118.1
	6×6×⅝	16×⅝	159.3	68.0	183.8
	6×6×⅝	14×¾	159.3	71.4	183.8
	6×4×¾	14×¾	156.9	71.4	118.1

m Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.
explanation of table, see page 706.

Table IV*† (Continued). Elements of Riveted Plate Girders

Section-modulus, axis 1-1, in³	Sizes: Web-plate, in	Sizes: Flange-angles, in	Sizes: Flange-plates, in	Weight per foot: Web-plate and flange-angles, lb	Weight per foot: Flange-plates, lb	Maximum end-reaction in thousands of pounds
1 129.9		6×6×⅝	16×¾	159.3	81.6	183.8
1 147.9		6×4×¾	16×¾	156.9	81.6	118.1
1 156.0		6×6×¾	14×¾	177.3	71.4	183.8
1 211.6	42×⅞	6×6×⅝	16×⅞	159.3	95.2	183.8
1 219 8		6×6×¾	16×¾	177.3	81.6	183.8
1 300.9		6×6×¾	16×⅞	177.3	95.2	183.8
1 387.3		6×6×⅞	16×⅞	194.9	95.2	183.8
513.5		6×4×½		136.2		135.0
563.5		6×6×½		149.8		210.0
597.7		6×4×⅝		151.4		135.0
659.8		6×6×⅝		168.2		210.0
678.4		6×4×¾		165.8		135.0
752.8		6×6×¾		186.2		210.0
755.8		6×4×⅞		180.2		135.0
799.2		6×4×½	14×½	136.2	47.6	135.0
841.7		6×4×½	16×½	136.2	54.4	135.0
842 7		6×6×⅞		203.8		210.0
848.1		6×6×½	14×½	149.8	47.6	210.0
870.8		6×4×½	14×⅝	136.2	59.5	135.0
890.6		6×6×½	16×½	149.8	54.4	210.0
919.4		6×6×½	14×⅝	149.8	59.5	210.0
952.6		6×4×⅝	14×⅝	151.4	59.5	135.0
972.6	42×½	6×6×½	16×⅝	149.8	68.0	210.0
990.8		6×6×½	14×¾	149.8	71.4	210.0
1 005 7		6×4×⅝	16×⅝	151.4	68.0	135.0
1 012.9		6×6×⅝	14×⅝	168.2	59.5	210.0
1 023.7		6×4×⅝	14×¾	151.4	71.4	135.0
1 066.0		6×6×⅝	16×⅝	168.2	68.0	210.0
1 083.7		6×6×⅝	14×¾	168.2	71.4	210.0
1 101.7		6×4×¾	14×¾	165.8	71.4	135.0
1 147.5		6×6×⅝	16×¾	168.2	81.6	210.0
1 165 4		6×4×¾	16×¾	165.8	81.6	135.0
1 173.6		6×6×¾	14×¾	186.2	71.4	210.0
1 229.0		6×6×⅝	16×⅞	168.2	95.2	210.0
1 237.4		6×6×¾	16×¾	186.2	81.6	210.0
1 318.4		6×6×¾	16×⅞	186.2	95.2	210.0
1 321.2		6×4×⅞	16×⅞	180.2	95.2	135.0
1 404.7		6×6×⅞	16×⅞	203.8	95.2	210.0
466.9		6×4×⅜		110.4		121.5
512.7		6×6×⅜		120.8		180.0
567.4	48×⅜	6×4×½		126.0		121.5
628.9		6×6×½		139.6		180.0
664.9		6×4×⅝		141.2		121.5

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.

† For explanation of table, see page 706.

Table IV *† (Continued). Elements of Riveted Plate Girders

Section-modulus, axis 1-1, in³	Sizes			Weight per foot		Maximum end-reaction thousands of pounds
	Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb	
714.4	48×3/8	6×4×3/8	14×3/8	110.4	35.7	121.5
741.3		6×6×5/8		158.0		180.0
750.8		6×4×3/8	16×3/8	110.4	40.8	121.5
758.5		6×4×3/4		155.6		121.5
759.5		6×6×3/8	14×3/8	120.8	35.7	180.0
795.9		6×6×3/8	16×3/8	120.8	40.8	180.0
797.0		6×4×3/8	14×1/2	110.4	47.6	121.5
841.9		6×6×3/8	14×1/2	120.8	47.6	180.0
848.3		6×4×7/8		170.0		121.5
850.1		6×6×3/4		176.0		180.0
890.4		6×6×3/8	16×1/2	120.8	54.4	180.0
895.5		6×4×1/2	14×1/2	126.0	47.6	121.5
924.3		6×6×3/8	14×5/8	120.8	59.5	180.0
944.0		6×4×1/2	16×1/2	126.0	54.4	121.5
955.2		6×6×7/8		193.6		180.0
955.8		6×6×1/2	14×1/2	139.6	47.6	180.0
977.7		6×4×1/2	14×5/8	126.0	59.5	121.5
1 004.3		6×6×1/2	16×1/2	139.6	54.4	180.0
1 037.6		6×6×1/2	14×5/8	139.6	59.5	180.0
1 072.7		6×4×5/8	14×5/8	141.2	59.5	121.5
1 098.2		6×6×1/2	16×5/8	139.6	68.0	180.0
1 119.5		6×6×1/2	14×3/4	139.6	71.4	180.0
1 133.3		6×4×5/8	16×5/8	141.2	68.0	121.5
1 147.1		6×6×5/8	14×5/8	158.0	59.5	180.0
1 154.4		6×4×5/8	14×3/4	141.2	71.4	121.5
1 207.8		6×6×5/8	16×5/8	158.0	68.0	180.0
1 228.4		6×6×5/8	14×3/4	158.0	71.4	180.0
1 245.2		6×4×3/4	14×3/4	155.6	71.4	121.5
1 301.2		6×6×5/8	16×3/4	158.0	81.6	180.0
1 317.9		6×4×3/4	16×3/4	155.6	81.6	121.5
1 334.0		6×6×3/4	14×3/4	176.0	71.4	180.0
1 394.7		6×6×5/8	16×7/8	158.0	95.2	180.0
1 406.7		6×6×3/4	16×3/4	176.0	81.6	180.0
1 498.1		6×4×7/8	16×7/8	170.0	95.2	121.5
1 499 7		6×6×3/4	16×7/8	176.0	95.2	180.0
1 601.3		6×6×7/8	16×7/8	193.6	95.2	180.0
591.2	48×7/16	6×4×1/2		136.2		141.8
652.7		6×6×1/2		149.8		210 0
688.7		6×4×5/8		151.4		141.8
765.0		6×6×5/8		168.2		210.0
782.3		6×4×3/4		165.8		141.8
872.1		6×4×7/8		180.2		141.8
873.8		6×6×3/4		186.2		210.0

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.
† For explanation of table, see page 706.

Table IV *† (Continued). Elements of Riveted Plate Girders

Section-modulus, axis 1-1, in³	Sizes: Web-plate, in	Sizes: Flange-angles, in	Sizes: Flange-plates, in	Weight per foot: Web-plate and flange-angles, lb	Weight per foot: Flange-plates, lb	Maximum end-reaction in thousands of pounds
918.8	48×7/16	6×4×½	14×½	136.2	47.6	141.8
967.3		6×4×½	16×½	136.2	54.4	141.8
979.0		6×6×⅞		203.8		210.0
979.0		6×6×½	14×½	149.8	47.6	210.0
1 000.8		6×4×½	14×⅝	136.2	59.5	141.8
1 027.6		6×6×½	16×½	149.8	54.4	210.0
1 060.8		6×6×½	14×⅝	149.8	59.5	210.0
1 095.8		6×4×⅝	14×⅝	151.4	59.5	141.8
1 121.4		6×6×½	16×⅝	149.8	68.0	210.0
1 142.5		6×6×½	14×¾	149.8	71.4	210.0
1 156.5		6×4×⅝	16×⅝	151.4	68.0	141.8
1 170.3		6×6×⅝	14×⅝	168.2	59.5	210.0
1 177.4		6×4×⅝	14×¾	151.4	71.4	141.8
1 230.9		6×6×⅝	16×⅝	168.2	68.0	210.0
1 251.5		6×6×⅝	14×¾	168.2	71.4	210.0
1 268.2		6×4×¾	14×¾	165.8	71.4	141.8
1 324.3		6×6×⅝	16×¾	168.2	81.6	210.0
1 341.0		6×4×¾	16×¾	165.8	81.6	141.8
1 357.0		6×6×¾	14×¾	186.2	71.4	210.0
1 417.7		6×6×⅝	16×⅞	168.2	95.2	210.0
1 429.8		6×6×¾	16×¾	186.2	81.6	210.0
1 521.0		6×4×⅞	16×⅞	180.2	95.2	141.8
1 522.7		6×6×¾	16×⅞	186.2	95.2	210.0
1 624.2		6×6×⅞	16×⅞	203.8	95.2	210.0
615.0	48×½	6×4×½		146.4		162.0
676.4		6×6×½		160.0		240.0
712.4		6×4×⅝		161.6		162.0
788.8		6×6×⅝		178.4		240.0
806.0		6×4×¾		176.0		162.0
895.8		6×4×⅞		190.4		162.0
897.6		6×6×¾		196.4		240.0
942.1		6×4×½	14×½	146.4	47.6	162.0
990.6		6×4×½	16×½	146.4	54.4	162.0
1 002.3		6×6×½	14×½	160.0	47.6	240.0
1 002.7		6×6×⅞		214.0		240.0
1 024.0		6×4×½	14×⅝	146.4	59.5	162.0
1 050.8		6×6×½	16×½	160.0	54.4	240.0
1 083.9		6×6×½	14×⅝	160.0	59.5	240.0
1 119.0		6×4×⅝	14×⅝	161.6	59.5	162.0
1 144.5		6×6×½	16×⅝	160.0	68.0	240.0
1 165.6		6×6×½	14×¾	160.0	71.4	240.0
1 179.6		6×4×⅝	16×⅝	161.6	68.0	162.0
1 193.4		6×6×⅝	14×⅝	178.4	59.5	240.0

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.
† For explanation of table, see page 706.

Table IV *† (Continued). Elements of Riveted Plate Girders

Section-modulus, axis 1-1, in³	Sizes			Weight per foot		Maxim end-reaction thousa of pound
	Web-plate, in	Flange-angles, in	Flange-plates, in	Web-plate and flange-angles, lb	Flange-plates, lb	
1 200.5	48×½	6×4×⅝	14×¾	161.6	71.4	162.
1 254.1		6×6×⅝	16×⅝	178.4	68.0	240.
1 274.5		6×6×⅝	14×¾	178.4	71.4	240.
1 291.2		6×4×¾	14×¾	176.0	71.4	162.
1 347.3		6×6×⅝	16×¾	178.4	81.6	240.
1 364.0		6×4×¾	16×¾	176.0	81.6	162.
1 380.0		6×6×¾	14×¾	196.4	71.4	240.
1 440.6		6×6×⅝	16×⅞	178.4	95.2	240.
1 452.8		6×6×¾	16×¾	196.4	81.6	240.
1 543.9		6×4×⅞	16×⅞	190.4	95.2	162.
1 545.6		6×6×¾	16×⅞	196.4	95.2	240.
1 647.1		6×6×⅞	16×⅞	214.0	95.2	240.

* From Pocket Companion, Carnegie Steel Company, Pittsburgh, Pa.
† For explanation of table, see page 706.

CHAPTER XXI

STRENGTH AND STIFFNESS OF WOODEN FLOORS

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The Problems Stated. The problems which are presented in this part of building-construction are, in general, (1) the designing of the joists and girders forming the framework of the floor to safely support the greatest load likely to come upon it, and (2) the determination of the maximum safe load for a floor already built. The first of these problems is the one with which architects and builders more commonly have to deal, and is, therefore, considered first.

Layout of the Floor-Framing. Before any calculations can be made for the sizes of the timbers it is necessary to know the spans of the joists, and, if there are openings in the floor, or the floor-joists have to support longitudinal partitions, a framing-plan should be made, showing the floor-area that will be supported by each joist, and also the position of partitions or special loads. If the floor is to be supported by posts and girders the position of these should also be accurately indicated on the framing-plan. Where the joists are supported entirely by walls or partitions, the spans of the joists will of course be fixed by the plan of the building. When the distance between a wall and a partition is too great for a single span, there may be a question as to the best positions for the posts and girders. When planning a building in which wooden joists are to be used, it is important to keep in mind the general scheme of the floor-framing and particularly the spans. Whenever practicable the spans of wooden joists should not exceed 24 ft. When the distance between the supporting walls exceeds 30 ft, girders should be placed so that the maximum span of the joists will not exceed 24 ft for light buildings nor from 16 to 18 ft for warehouses.

In School Buildings it is desirable to have the rooms at least 27 ft wide, and since in this class of buildings the joists usually have spans of from 27 to 30 ft. For a span of 30 ft, however, 16-in joists should be used, and as these are expensive, and often difficult to obtain, it is much better and more economical to make the schoolrooms 27 by 32 or 34 ft, than to make them 30 ft square. A schoolroom 27 ft wide by from 32 to 34 ft long, with windows on the long side, is economical and satisfactory, as it permits of using 3 by 14-in joists, 28 ft long, and also results in the most satisfactory lighting.

Continuous Joists. When joists are supported by a girder placed so that a 24 or 26-ft joist extends over the two spans, it is always better to have the joists continuous over the girder, as by that construction they make a much stiffer floor. (See Chapter XIX.)

Floor-Loads. Having decided on the arrangement of the joists, and drawn a framing-plan showing the span and the locations of all special timbers, the

next step involves the determination of the loads for which the joists and girders are to be proportioned. Floor-loads are made up of two parts, the weight of the materials composing the floor itself, and the ceiling below, if there is one; and the load liable to be put on the floor. The first is called the DEAD LOAD, and the second the LIVE LOAD. When the SAFE LOAD for a floor is spoken of the live load is generally meant.

Weight of Wooden Floor-Construction. Wooden floors usually consist of (1) beams, commonly called JOISTS,* or FLOOR-JOISTS, (2) one or two thicknesses of flooring-boards, and, in a finished building, (3) a ceiling underneath the joists. In figuring the weight of ⅞-in flooring-boards it will be sufficiently accurate to estimate the weight of a single thickness at 3 lb per sq ft. joists may also be figured at 3 lb per ft, board-measure, with the exception of hard-pine and oak joists, which should be figured at 4 lb per ft, board-measure. The weight of the joists must also be reduced to their equivalent weight per square foot of floor. Thus, the weight of a 2 by 12-in joist is about 6 lb per ft. If the joists are spaced 12 in on centers, this will be equal to 6 lb per sq ft; but if the joists are 16 in on centers there will be but one linear foot of joist to every 1⅓ sq ft, which will be equivalent to 4½ lb per sq ft; and if they are 20 in on centers, the weight will be equal to 3⅗ lb per sq ft; spaced 24 in on centers, the weight will be 3 lb per sq ft. The weight of a lath-and-plaster ceiling should be taken at 10 lb per sq ft, and of a ¾-in wooden ceiling at per sq ft. A corrugated-iron ceiling weighs about 1 lb per sq ft. For stamped steel ceilings, 2 lb per sq ft will cover the weight of the metal and furring. The following table, giving the weight of joists, will be found convenient in figuring the weight of floors:

Table I. Weight of Floor-Joists per Square Foot of Floor

Sizes of joists	Spruce, hemlock, white pine. Spacing in inches, center to center		Hard pine or oak. Spacing in inches, center to center	
	12	16	12	16
in	lb	lb	lb	lb
2×6	3	2¼	4	3
2×8	4	3	5⅓	4
3×8	6	4½	8	6
2×10	5	3¾	6⅔	5
3×10	7½	5⅝	10	7½
2×12	6	4½	8	6
3×12	9	6¾	12	9
2×14	7	5¼	9⅓	7
3×14	10½	8½	14	10½

Weight of Crowds. L. J. Johnson reports† results of some tests to tain the weight of crowds of men, in which he obtained weights of 134.2,

* Some building laws use the term FLOOR-BEAM instead of the word JOIST.

† See Engineering News, April 14, 1904.

1.1 and 156.9 lb per sq ft. The last-mentioned weight was obtained by packing 67 men in a room about 6 by 11 ft in size. Professor Johnson also found that with 50 men in the room, making a load of 122 lb per sq ft, the crowd was compacted "so that a man could elbow his way through it only with perseverance and determined effort."

Superimposed Loads. There is much difference of opinion as to what allowance should be made for the live load. Table II shows the minimum allowance for live loads for different classes of buildings, as fixed by the building laws of the cities mentioned. (See, also, page 149.)

Table II. Minimum Safe Superimposed Loads for Floors, Required by Various Building Laws

Classes of buildings	Minimum live load per square foot of floor					
	Buffalo, 1905	Boston, 1915	Chicago, 1916	Philadelphia, 1914	New York, 1917	St. Louis, 1910
Dwellings	40	50	40	70	40	60
Hotels, tenements and lodging-houses	70	50	50	70	60	60
Office-buildings	70	80	50	100	60*	70*
Buildings for public assembly	100	100	100	120	100	100†
Stores, warehouses and mfg. bldgs	120‡	125‡	100‡	120‡	120‡	150‡

* First floor, 150 lb. † Also school-houses. ‡ And upwards.

It was the opinion of Mr. Kidder that the following allowances for floor-loads, taken in connection with the values given for the safe strength of joists or beams, provide absolute safety with proper allowance for economy.

	Lb per sq ft
For dwellings, sleeping-rooms and lodging-rooms	40
For schoolrooms	50
For office-buildings, upper stories	60
For office-buildings, first story	80
For stables and carriage-houses	65
For banking-rooms, churches and theaters	80
For assembly-halls, dancing-halls and the corridors of all public buildings, including hotels	120
For drill-rooms	150

Live Loads for Stores and Buildings for Light Manufacturing. Floors for ordinary stores, light manufacturing and light storage should be computed for not less than 120 lb per sq ft, and for a concentrated load at any point of ...0 lb.

Live Loads for Dwellings, etc. Floors of dwellings, tenements, lodging-houses and rooms in hotels, are seldom loaded with more than 20 lb per sq ft for the entire area, and a minimum load of 40 lb per sq ft should provide for all possible contingencies.

Live Loads for Office-Buildings. The floors of offices are, as a rule, not m heavily loaded than the floors of dwellings, but the possibilities for increa loads from safes and heavy furniture, and possibly from a more compact cro of people, are greater, so that the minimum floor-load for offices should be so what increased. Some years ago the firm of Blackall & Everett, in Bos found that the average live load in 210 offices, in three prominent office-buildi in that city, was between 16 and 17 lb per sq ft, while the average load for the heaviest office-buildings was 33.3 lb per sq ft. As such loads, however, are a rule unevenly distributed, some portions of the floor being generally m more heavily loaded than others, it would not appear to be safe to use t average to determine the strength of floor-beams and floor-arches, although would probably answer for the columns. There seems to be a considerable diff ence of opinion among the leading architects and structural engineers as to j what allowance should be made for office-floors. Among some of the ear fire-proof office-buildings, for example, may be mentioned the former M Building in San Francisco in which the live loads were assumed at 40 lb per ft for all floors above the first. In the Venetian Building, Chicago, the seco third and fourth floors were calculated for 60, and the upper floors for 35 lb sq ft of live load, while in the Old Colony and Fort Dearborn Buildings Chicago, the live loads on the floor-beams were assumed at 70 lb per sq At the present time (1915), 50, 60, 70, 75, 100 and 150 lb per sq ft are minimum live loads for the design of floors of office-buildings required by building laws of six different cities. C. C. Schneider recommends* for design of floors of office-buildings above the first floor, for the uniform l of the floor-area, 50; for concentrated loads applied at any point of the fl 5 000; and for the uniform load for girders, 1 000; the 50 being in pounds sq ft, the 5 000 in pounds and the 1 000 in pounds per linear foot.

Live Loads for Churches, Theaters and School-Houses. "An allowance 120 lb per sq ft for the live load in churches, theaters and school-houses is m greater than the actual conditions require. The average size of a schoolro is about 28 by 32 ft, and such a room usually contains seats for fifty-six scho and the teacher. Assuming the average weight of each scholar at 120 lb, average live load, including ten visiting adults and the desks and furniture 13 lb per sq ft. Even supposing that the scholars of two rooms were uni for some special occasion, there would be only 22 lb per sq ft; and this is great a load as it is possible to imagine in such a room, as the fixed desks prev the crowding together of the scholars except at the sides of the room. F this reasoning, therefore, 50 lb per sq ft would appear ample for schoolroo As a matter of fact, 3 by 14-in long-leaf yellow-pine joists, 16 in on cen and with a 28-ft span, have been used for school-room floors for years; such beams, if calculated by the formula for stiffness, would support a load of only 43 lb per sq ft. (Table XII, page 643 and Table I, page 7 The minimum floor-space allotted to a single seat in theaters is 4 sq ft, w the average is about 5 sq ft. Assuming the weight of an opera-chair at 3 and of the average adult at 140 lb, a liberal allowance, there results an ave of 44 lb per sq ft of floor. A minimum of 80 lb per sq ft would therefore s to provide for any possible crowding during a panic, except in corridors. the other hand, it has been shown (see Weight of Crowds, page 718) th crowd of able-bodied men may result in a load of about 120 lb per sq ft, this should be the minimum for assembly-halls without fixed desks and also the corridors of all public buildings. For armories, the minimum load sh be increased on account of the vibration."†

* "General Specifications for Structural Work of Buildings," 1910, page 53.
† F. E. Kidder.

The Average Floor-Loads for Stores has also been greatly over-estimated. W. L. B. Jenney found that the average load on the floors of the wholesale warehouse of Marshall Field & Company, in Chicago, was but 50 lb per sq ft, and very few retail stores will average over 80 lb per sq ft. An allowance of 120 lb per sq ft is sufficient for ordinary retail stores, with the possible exception of hardware stores.

Live Loads for Warehouses. Warehouses, on the other hand, may be very heavily loaded, and the floors in buildings intended for the storage of merchandise should be proportioned to the especial class of goods which they are designed to support. Table III, originally compiled by C. J. H. Woodbury,* and to which some additions have been made by the Insurance Engineering Experiment Station and by Mr. Kidder, will be found of assistance in deciding upon the live load to be assumed for warehouse-floors. The weights per square foot are for single packages. If the goods are piled two or more cases high, the weight per square foot of floor will of course be increased accordingly. In fact, the height to which the goods are liable to be piled is a very important consideration in fixing upon the floor-load. In Table III "the measurements were always taken to the outside of case or package, and gross weights of such packages are given."

Methods of Determining the Sizes of Joists, Beams or Girders Required for Any Building. As already explained, the first step is the making of a framing-plan of the floors or enough of it to show any special framing and also the span and floor-area supported by the different joists, beams or girders.

Table III. Weights of Merchandise †

Materials	Measurements		Weights in pounds		
	Floor-space, sq ft	Contents, cu ft	Total, lb	Per sq ft	Per cu ft
Wool					
Bale, East India	3.0	12.0	340	113	28
Bale, Australia	5.8	26.0	385	66	15
Bale, South America	7.0	34.0	1 000	143	29
Bale, Oregon	6.9	33.0	482	70	15
Bale, California	7.5	33.0	550	73	17
Bag. wool	5.0	30.0	200	40	7
Sack of scoured wool	.....	......	......	......	5
Woollen Goods					
Case, flannels	5.5	12.7	220	40	17
Case, flannels, heavy	7.1	15.2	330	46	22
Case, dress goods	5.5	22.0	460	84	21
Case, cashmeres	10.5	28.0	550	52	20
Case, underwear	7.3	21.0	350	48	16
Case, blankets	10.3	35.0	450	44	13
Case, horse-blankets	4.0	14.0	250	63	18

* The Fire Protection of Mills, page 118. † See, also, pages 1501 to 1508.

Table III (Continued). Weights of Merchandise

Materials	Measurements		Weights in pounds		
	Floor-space, sq ft	Contents, cu ft	Total, lb	Per sq ft	Per cu ft
Cotton, Etc					
Bale	8.1	44.2	515	64	12
Bale, compressed	4.1	21.6	550	134	25
Bale, American Cotton Co.	4.0	11.0	263	66	24
Bale, Planters' Compressed Co.	2.3	7.2	254	110	35
Bale, jute	2.4	9.9	300	125	30
Bale, jute lashings	2.6	10.5	450	172	43
Bale, manila	3.2	10.9	280	88	26
Bale, hemp	8.7	34.7	700	81	20
Bale, sisal	5.3	17.0	400	75	24
Cotton Goods					
Bale, unbleached jeans	4.0	12.5	300	72	24
Piece duck	1.1	2.3	75	68	33
Bale, brown sheetings	3.6	10.1	235	65	23
Case, bleached sheetings	4.8	11.4	330	69	30
Case, quilts	7.2	19.0	295	41	16
Bale, print cloth	4.0	9.3	175	44	19
Case, prints	4.5	13.4	420	93	31
Bale, tickings	3.3	8.8	325	99	37
Skeins, cotton yarn	......	......	......	......	11
Burlaps	......	......	130	......	30
Jute bagging	1.4	5.3	100	70	24
Rags in Bales					
White linen	8.5	39.5	910	107	23
White cotton	9.2	40.0	715	78	18
Brown cotton	7.6	30.0	442	59	15
Paper shavings	7.5	34.0	507	68	15
Sacking	16.0	65.0	450	28	7
Woollen	7.5	30.0	600	80	20
Jute butts	2.8	11.1	400	143	36
Paper					
Calendered book	......	......	......	......	[illegible]
Supercalendered book	......	......	......	......	[illegible]
Newspaper	......	......	......	......	[illegible]
Strawboard	......	......	......	......	[illegible]
Leather-board	......	......	......	......	[illegible]
Writing	......	......	......	......	[illegible]
Wrapping	......	......	......	......	[illegible]
Manila	......	......	......	......	[illegible]

Table III (Continued). Weights of Merchandise

Materials	Measurements		Weights in pounds		
	Floor-space, sq ft	Contents, cu ft	Total, lb	Per sq ft	Per cu ft
Grain *					
Wheat, in bags	4.2	4.2	165	40	40
Wheat, in bulk, mean	......	......	......	......	48
Barrels, flour, on side	4.1	5.4	218	53	40
Barrels, flour, on end	3.1	7.1	218	70	31
Corn, in bags	3.6	3.6	112	31	31
Cornmeal, in barrels	3.7	5.9	218	59	37
Oats, in bags	3.3	3.6	96	29	27
Bale of hay	5.0	20.0	284	57	14
Hay, Dederick, compressed	1.75	5.25	125	72	24
Straw, Dederick, compressed	1.75	5.25	100	57	19
Tow, Dederick, compressed	1.75	5.25	150	86	29
Excelsior, Dederick, compressed	1.75	5.25	100	57	19
Hay, loose	......	......	......	......	4
Dyestuffs, Etc					
Hogshead, bleaching powder	11.8	39.2	1 200	102	31
Hogshead, soda-ash	10.8	29.2	1 800	167	62
Box, indigo	3.0	9.0	385	128	43
Box, cutch	4.0	3.3	150	38	45
Box, sumac	1.6	4.1	160	100	39
Caustic soda in iron drum	4.3	6.8	600	140	88
Barrel, starch	3.0	10.5	250	83	24
Barrel, pearl-alum	3.0	10.5	350	117	33
Box, extract logwood	1.06	0.8	55	52	70
Barrel, lime	3.6	4.5	225	63	50
Barrel, cement, American	3.8	5.5	325	86	59
Barrel, cement, English	3.8	5.5	400	105	73
Barrel, plaster	3.7	6.1	325	88	53
Barrel, rosin	3.0	9.0	430	143	48
Barrel, lard-oil	4.3	12.3	422	98	34
Rope	......	......	......	......	42
Miscellaneous					
Box, tin	2.7	0.5	139	99	278
Box, glass	......	......	......	......	60
Crate, crockery	9.9	39.6	1 600	162	40
Cask, crockery	13.4	42.5	600	52	14
Bale, leather	7.3	12.2	190	26	16
Bale, goatskins	11.2	16.7	300	27	18
Bale, raw hides	6.0	30.0	400	67	13
Bale, raw hides, compressed	6.0	30.0	700	117	23
Bale, sole-leather	12.6	8.9	200	22	16
Pile, sole-leather	......	......	......	......	17
Barrel, granulated sugar	3.0	7.5	317	106	42
Barrel, brown sugar	3.0	7.5	340	113	45
Cheese	......	......	......	......	30

* For pressure of grain in deep bins, see Engineering News, March 10, 1904, pages 2[illegible] and 336, and Dec. 15., 1904.

The second step is to determine approximately the weight of the floor and ce ing, and decide what superimposed load per square foot the floor is to be design to carry. Having done this, the next step is the computing of the requir dimensions of the common floor-joists. For most buildings the size of floo joists required can be readily determined by reference to Tables XIII to XV inclusive, and XXII to XXVI, inclusive, of this chapter. For other floor-loa the sizes of the common joists may be determined by computing the load be supported by a single joist and then, by the formulas or tables in Chap XVI or the formulas in Chapter XVIII, determining the dimensions of the joi to support that load. (See Example 1.) For the floors of all buildings exce stores and warehouses it is recommended that the sizes of the common joi be determined by the formulas for stiffness in Chapter XVIII or the stiffne values in the tables in Chapter XVI, unless one value, only, is given in tab for safe loads, in which case that value may be used. For stores and wa houses the sizes of the joists may be proportioned by the formulas or streng values of the tables in Chapter XVI.

The Dimensions of Special Beams, such as headers, trimmers and beams su porting partitions, and also of the girders, should be determined in the sa way, that is, by comp ing the maximum lo the beam may have support, and then t dimensions of a be that will support th load with safety. T manner of making computations is e plained in the follow examples.

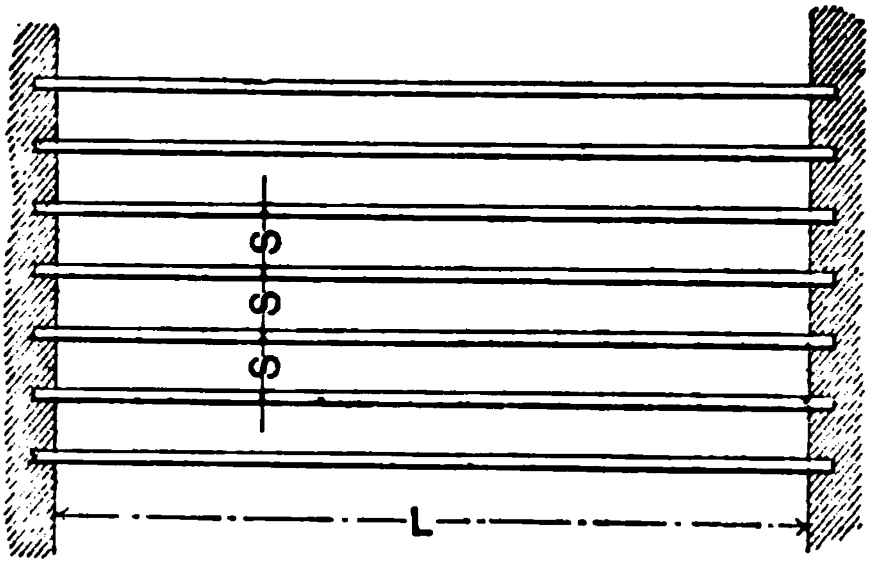


Fig. 1. Plan of Floor-joists

Example 1. The si plest type of floor-fra ing is that shown in F 1, in which all of joists are of the sa span and support equal floor-areas. In such a floor, the FLOOR-AREA su ported by each joist is equal to the span, L, multiplied by the spacing, S, feet. The LOAD on each joist is equal to the FLOOR-AREA multiplied by sum of the dead-loads and superimposed or live loads. To show the appli tion of the above-mentioned formulas and tables we will assume that Fig represents the framing of a floor in a dwelling-house or lodging-house, t $L = 18$ ft, $S = 16$ in or 1⅓ ft, and that the timber is common white pine. T joists are to support a plastered ceiling and a double floor of ⅞-in boar What should be the size of the joists; average quality, conditions not ideal?

Solution. The FLOOR-AREA supported by each joist is 1⅓ by 18, or 24 sq From Table XIII or XXII, pages 737 and 742, for a span of 18 ft, the jo will probably have to be at least 2 by 12 in, and their weight will be about 4½ per sq ft (see Table I, page 718). The plastered ceiling weighs about 10 lb the flooring 6 lb per sq ft, making the total weight of the floor 20½ lb per sq For the superimposed load we should allow at least 40 lb per sq ft (see p 719). This might be greater, if exacted by any particular building law. load on a single joist will, therefore, be, with these assumed unit loads, 60½ by 24 sq ft, or 1 452 lb.

From Table VIII, page 639, we find that the maximum load for a 1

in white-pine joist of 18 ft span is 623 lb; hence to support 1 452 lb will require a breadth equal to 1452/623 = 2⅓ in. Therefore, to comply with the requirements for both strength and stiffness, the joists should be 2⅓ by 12 in. This is not a stock size. Joists 2 by 12 in, 12 in on centers, may next be tried. Each joist must support 1 116 lb, requiring, by Table VIII, page 639, a 1.8 by 12-in joist, determined by the quotient 1 116/623. So that, in this example, white-pine joists of a nominal size of 2 by 12 in and spaced 12 in on centers might be used, although they are slightly under the required depth, as the dressed size is about 1¾ by 11½ in. From Table VI, page 637, the conversion-factor is 1.61, and 623 lb × 1.61 = 1 003 lb which is less than 1 116 lb, the load to be supported. From Tables XIII and XXII, pages 737 and 742, the maximum spans for 2 by 12 in white-pine joists, 12 in on centers, are 19 ft and 18 ft 8 in respectively, according to the assumed value of the modulus of elasticity for white pine. For 3 by 12-in joists, 16 in on centers, the load is 1 506 lb, and 1 506/623 = 2⅖ in. The dressed size is almost 2¾ by 11½ in, the conversion-factor, 2.53, and 623 × 2.53 = 1 576 lb, an amount greater than 1 506 lb. Tables XIII and XXII, again, give 19 ft 8 in and 19 ft in for the maximum spans. Joists 3 by 12, 16 in on centers, are stronger than necessary. If, in this example, the span is made ft, by Table VIII, page 639, for 12-in joists two values for the safe loads are found, and the smaller, stiffness-value, should be used, unless the deflection need not be considered.

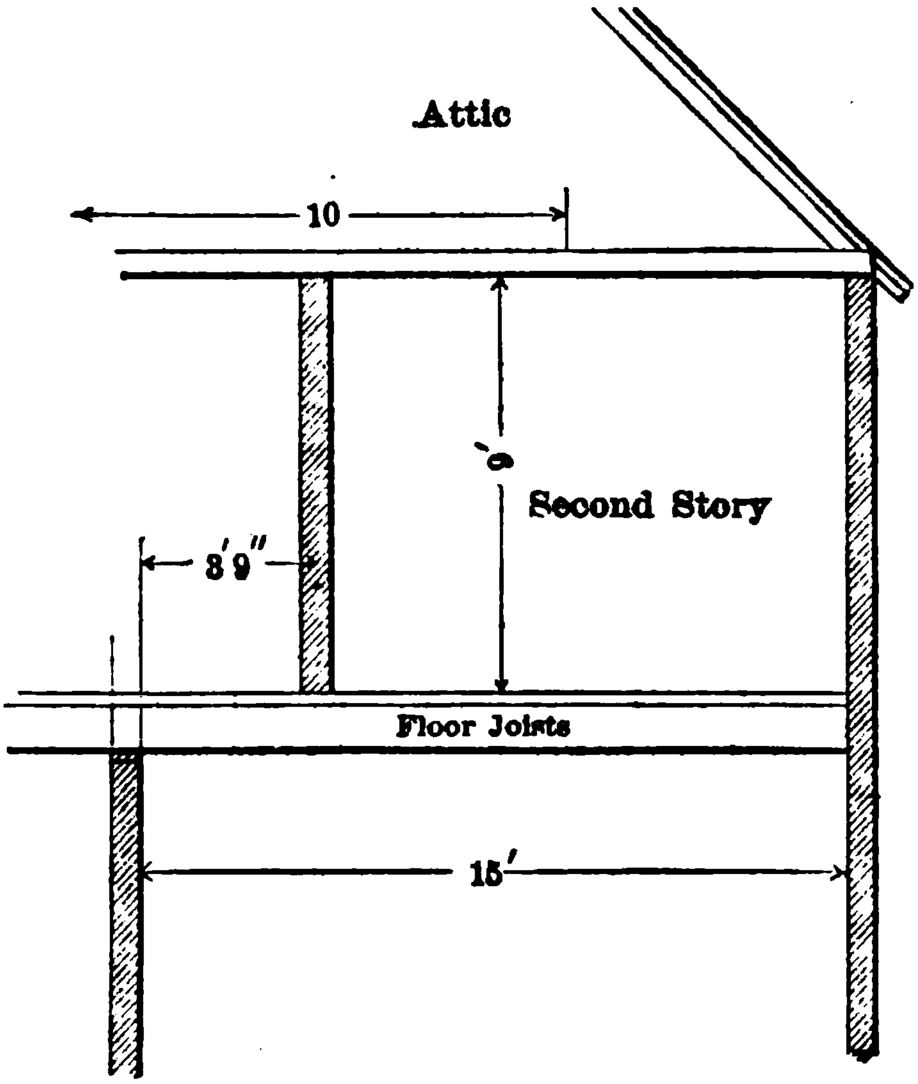


Fig. 2. Section Through Floors and Partitions

Example 2. Fig. 2 shows a partial section of a dwelling, in which the second-floor joists support a plastered partition which also supports the attic joists. What should be the size of the second-floor joists to meet the requirements of STRENGTH, the timber being fair-quality Eastern spruce with a safe fiber-stress assumed to be 700 lb per sq in for flexure? As the effect of a concentrated load, compared with a distributed load, in producing deflection, is not as great as the comparative effect in producing rupture, whenever a beam has a considerable CONCENTRATED load it may be calculated by the formula or tables FOR STRENGTH ONLY. The timber is assumed to be poorly seasoned.

Solution. The first step will be to determine the load on a single floor-joist. We will assume, as a trial, that the joists are to be 2 by 10 in, 12 in on centers, that both the first-story and second-story ceilings are to be plastered, and that

only single flooring will be used in the second story and attic. We will assu that the attic-joists are to be 2 by 8 in, 16 in on centers, and that the width floor supported by the partition is 10 ft.

The second-floor area supported by a single joist is 12 in by 15 ft, or 15 sq The weight of the floor-joists per sq ft is 5 lb, of the plastered ceiling 10 lb a of the flooring 3 lb, making the dead load per sq ft 18 lb. For the live or sup imposed load we should allow 40 lb and hence the load per square foot on ea second-floor joist due to the second floor and its load is 58 lb. As the flo area for a single joist is 15 sq ft the load from the second floor is 15 sq ft 58 lb per sq ft or 870 lb on each joist. We must now find what will be t load from the partition and attic-floor. The attic-floor and ceiling weigh abo 16 lb per sq ft, and 24 lb is a sufficient allowance for the live load. The weig per linear foot on the partition will therefore be 400 lb. A partition of 2 4-in studs, lathed and plastered on both sides, weighs about 20 lb per sq ft face; hence the partition itself weighs 180 lb per lin ft. The partition a attic-floor, therefore, bring a load of 580 lb on each second-floor joist, co CENTRATED at a point ONE-FOURTH of the span from the inner end of the joi To combine this concentrated load with the load from the second floor, must multiply the concentrated load by 1.5 (Table IV, page 632), which gi an equivalent distributed load of 870 lb. Adding this to the second-floor lo we have 1 740 lb as the total load for which each joist should be proportion From Table VIII, page 639, we find that the safe load for a 1 by 10-in spr joist of 15-ft span is 518 lb; hence the breadth of each joist should be equal 1 740/518 = 3.36 or about 3½ in. Deeper joists, therefore, must be used. we try 2 by 12-in joists, 12 in on centers, the safe load for a 1 by 12-in spr joist of 15-ft span is 747 lb. Hence the breadth is 1 755/747 = 2.35 or abo 2⅜ in, indicating 2⅜ by 12-in joists, 12 in on centers. If the fiber-stres assumed at 800 lb per sq in, the values of Table X, page 641, may be us This will give, for 2 by 12-in joists, 12 in on centers and 15-ft span, 854 lb the safe load for a 1 by 12-in joist; and 1 755/854 = about 2 in. The load p sq ft on each of these joists is 1 755/15 = 117 lb; and Tables XVI and XX pages 739 and 744, give 16 ft 6 in and 16 ft 1 in for the maximum s spans.

Example 3. It is required to determine the sizes of the girders and joists the floor shown in Fig. 3, all of the timbers being of long-leaf yellow pine, a the floor above being supported by posts and girders in the same way. T building is intended for lodging purposes, and the height of the story is 10 There is to be a double floor and the ceilings and partitions are to be plaster The floor-joists are to be spaced 16 in on centers. Average timber, poorly seasor

Solution. We will first determine the size of the common joists at *A*, call the span 24 ft. The floor-area supported by a single joist is 24 by 1⅓ ft, 32 sq ft.

From Table XIII or XXII, pages 737 and 742, for a 24-ft span, 2½ by 14 joists are probably required. We will allow 8¾ lb per sq ft for the weight joists and bridging (Table I, page 718), 10 for the ceiling and 6 for the floori making 24¾ lb per sq ft for the dead load. For the live load we will allow lb per sq ft. The load for which the joists should be proportioned is, th fore, 32 by 64¾ or 2 072 lb. We may use Table XII, page 643, to find maximum load for a 1 by 14-in joist of 24-ft span. The deflection-load give the table is 882 lb; hence the thickness of the joists must equal 2 072/88 2.35 or about 2⅜ in. Therefore 2½ by 14-in long-leaf yellow-pine joists, 16 on centers, may be used, but they should run full 2½ in thick. The jo at *B* (Fig. 3) have to support a partition, but as the span is much less, and

partition is quite near the end of the joists, it will be safe to make them of the same size as at *A*.

The joists at *C* (Fig. 3) have the same floor-load to support as at *A*, and in addition the weight of the partition, which is concentrated at one-third of the span from one support. As the partition is 10 ft high, 13⅓ sq ft of partition will be supported by each joist, the joists being 16 in on centers. Assuming 20 lb

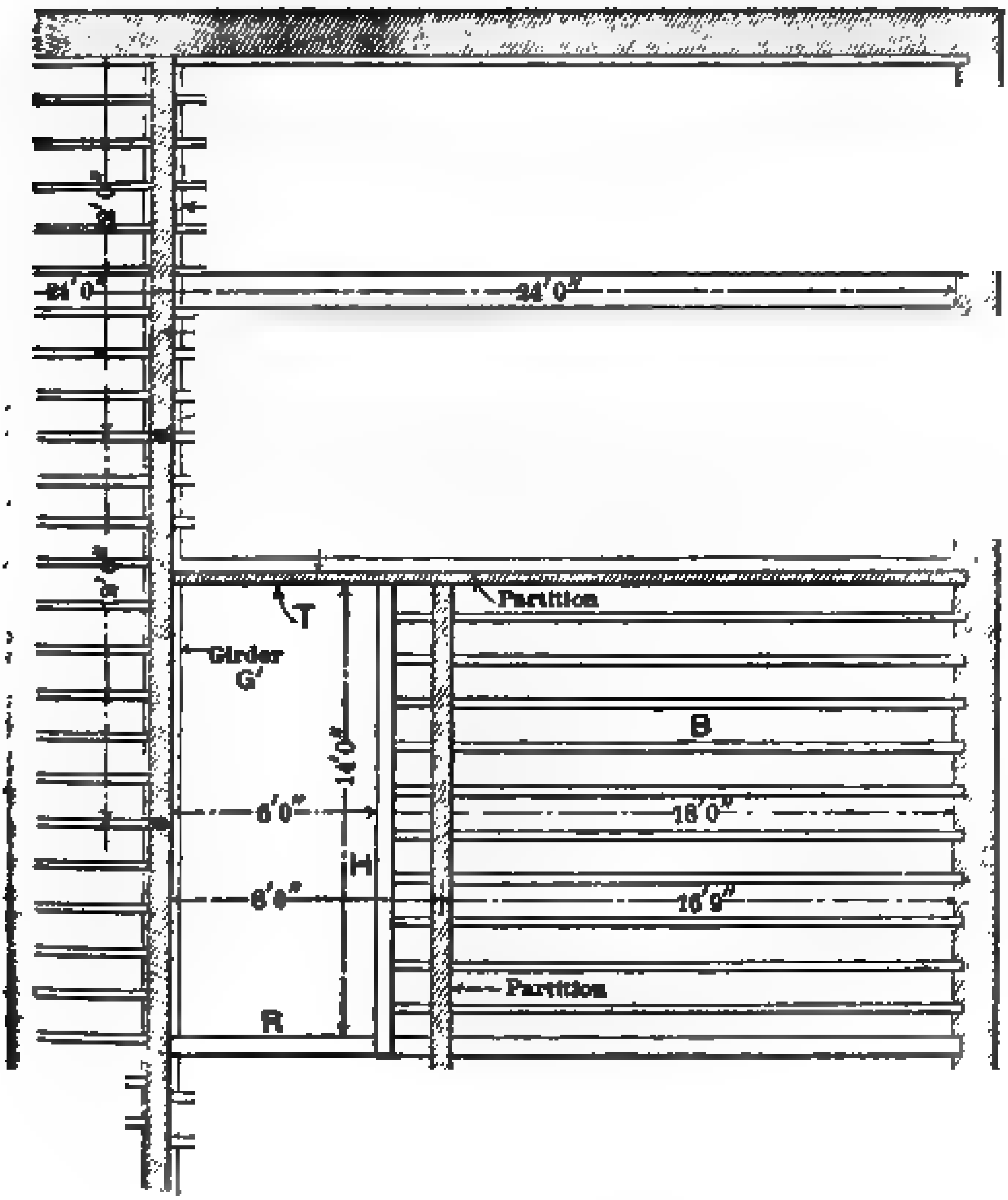


Fig. 3. Plan of Floor-framing Showing Partitions Above

sq ft of face as the weight of the partition, we have 267 lb as the weight the partition to be borne by each joist. To reduce this to an equivalent tributed load, we should multiply by 1.78 (Table IV, page 632), which gives lb. The joists at *C*, therefore, should be proportioned to a uniformly tributed load of 2 072 + 468 = 2 540 lb, which requires 14-in joists, 2.88 in r, say, 3 by 14-in joists.

The Header. We will next determine the required breadth for the head H (Fig. 3), the depth being necessarily 14 in, the same as for the joists.

The header is 14 ft long and must support the floor half-way to the wall, a floor-area of 14 by 9 ft, or 126 sq ft. Multiplying this area by 64¾ lb, t weight per square foot, we have 8 159 lb, the total floor-load to be support to which must be added a certain percentage of the partition. The portion the partition supported by the header is (14 ft − 1 ft 4 in) = 12 ft 8 in long a 10 ft high, and will weigh about 20 lb per sq ft of face, or a total of 2 532 As the partition is one-ninth of the span from the header, eight-ninths of weight will be supported by the header and one-ninth by the wall. Eight-nin of 2 532 is 2 251 lb, which, added to the floor-load, makes a total load the header of 10 410 lb. From Table XII, page 643, we find that the s load for a 1 by 14-in beam of long-leaf yellow pine, 14-ft span, is 1 867 lb; he it will require a breadth of 10 410/1 867 = 5.58 in. If the tail-beams are fram into the header, it should be thicker to allow for the weakening effects of t framing; so that, in this case, the header should be at least 6 by 14 in in act cross-section, before any framing is done.

The Trimmers. We will next consider the trimmer, T (Fig. 3). T beam has four loads: (1) a distributed floor-load; (2) a distributed load fr the partition above; (3) one-half the load on the header H; (4) and a sm direct load from the longitudinal partition.

(1) The strip of floor supported by the trimmer will be about 12 in wide a 24 ft long, and will weigh 64¾ lb per sq ft × 24 sq ft = 1 554 lb.

(2) The partition above will weigh 10 × 24 ft × 20 lb per sq ft = 4 800 lb

(3) One-half of the load on H is 10 410/2 = 5 205 lb. As this is concentra at one-fourth the span from the support, we must multiply it by 1.5 (Table page 632) to obtain the equivalent distributed load, which then becor 5 205 × 1.5 = 7 808 lb.

(4) About 8 in of the longitudinal partition must be supported by the trimm and this will weigh 10 × ⅔ ft × 20 lb per sq ft = 133 lb. As it is concentra at one-third the span from the support, we must multiply by 1.78 (Table page 632) to obtain the equivalent distributed load, which then becor 133 × 1.78 = 237 lb.

The total load for which the trimmer must be computed will be, therefore

(1) From the floor	1 554 lb
(2) From the partition above	4 800 lb
(3) From the header	7 808 lb
(4) From the longitudinal partition	237 lb
Total	14 399 lb

The trimmer should be of the same depth as the joists, 14 in. From Ta XII, page 643, we find that a 1 by 14-in long-leaf yellow-pine beam of 2 span will safely support 882 lb and not cause a deflection of more than 1⁄360 the span. Hence, the breadth of the trimmer would be 14 399/882 = 16.34 which is greater than the depth. This would suggest the substitution of a s I beam of proper size or the use of a deeper wooden beam, such as an 11 by 1 a 12 by 16-in beam. If the deflection of the wooden beam is not taken i account, the strength-value, 1 090 lb of Table XII, page 643, may be u giving 14 399/1 090 = 13.21 in as the width of the beam. This would ag with the former New York Code for strength. If the flexure fiber-stre taken at 1 300 lb per sq in, permitted by the Chicago code, Table XIII, p 644, may be used, giving 14 399/1 179 = 12.21 in for the width of the trimr

h 800 lb per sq in is taken for S, Table XV, page 646, is used, giving 14 399/ 633 = 8.81 in for the width. Hence, the architect will be governed by laws ities, or by engineering judgment or experience elsewhere, and this applies the joists as well as to the girders. If wooden trimmers are used, they should hung in beam-hangers (see last part of this chapter). The load on the trim- r, R, will be the same as on the trimmer, T, except for the cross-partition. ducting the weight of this partition, we have 14 399 − 4 800 = 9 599 lb for e equivalent distributed load on R, which, from Table XII, page 643, gives, for e required breadth 10.88 in or 8.8 in, depending upon whether the deflection or is not considered. Other variations in the required width of a 14-in oden girder will result from the use of other fiber-stresses.

The Girders. The floor-area supported by the girder, G (Fig. 3), is equal to by 24 ft, or 288 sq ft. As a general rule, it will be safe in estimating the live d on girders to take only 85% of the load assumed for the floor-beams, be- use there will always be some portion of the floor supported by the girder that not loaded, and probably other portions that will not be loaded up to the sumed load. Hence, the live load would be 85% of 40 lb, or 34 lb. The d load of the floor and ceiling will be about 25 lb, and the girder itself will gh between 1 and 2 lb per sq ft, say 2 lb per sq ft of floor, more, so that we l use 61 lb per sq ft for the total floor-load on this girder. As girder G sup- rts 288 sq ft, this will be equivalent to 17 568 lb. The girder supports, also, partition, 9 ft high, above, which will weigh 12 × 9 × 20 = 2 160 lb. The total d for which the girder should be proportioned is, therefore, 19 728 lb. As- ming 14 in for the depth of the girder, we find from Table XII, page 643, t the safe load for a 1 by 14-in long-leaf yellow-pine beam of 12-ft span is 867 lb; hence the breadth of girder, G, should be 19 728/1 867 = 10.56 in and 11 by 14-in girder could be used.

The girder, G' (Fig. 3), supports a floor-area at the left of 12 × 12 = 144 sq ft, ich represents a distributed load of 8 784 lb. On the right side of the girder re is a strip of floor 40 in wide by 12 ft long (8 in of the floor being included the load on T) which will weigh 2 440 lb. This may be considered as a con- trated load applied 20 in, or one-seventh the span, from the end of the der, in which case the effect of the load is practically the same as if the load re distributed. The load coming upon girder G' from T will equal one-half e actual distributed load on T, plus three-eighths (½ of ¾) of the load on H. e load on H we found to be 10 410 lb, and three-eighths of this is about 00 lb. The actual distributed load on T we found to be 1 554 + 4 800 = 54 lb, and one-half of this is 3 177 lb. Hence the trimmer, T, transmits a d of 3 900 + 3 177 = 7 077 lb to the girder, which must be considered as a centrated load applied at one-third the span from the support, and hence we t multiply it by 1.78 (Table IV, page 632) to obtain the equivalent dis- uted load, which gives 12 597 lb.

The load for which the girder, G' (Fig. 3), should be computed will be

From the floor at the left	8 784 lb
From the floor at the right	2 440 lb
From the trimmer, T	12 597 lb
From the partition above	2 160 lb
Total	25 981 lb

om Table XII, page 643, we find that this load will require a (13.9 by) 14 by 14-in girder. For this floor, therefore, the requirements, if -leaf yellow pine is used, and if the maximum flexure fiber-stress, S, is

taken at 1 200 lb per sq in (a conservative value for non-ideal conditio for example) and the modulus of elasticity, E, at 1 500 000 lb per sq in, are follows: an 11 by 14-in girder at G; a 14 by 14-in girder at G'; an 11 by 16, or by 16-in wooden beam or a steel I beam for the trimmer, T; an 11 by 14 beam for the trimmer, R; a 6 by 14-in beam for the header, H; 2½ by 14 joists at A and B; and 3 by 14-in joists at C. For these stress-requireme the architect might decide to use steel I beams for girders G, G', etc., and the trimmers, T and R. For S, 1300, Table XIII, page 644, may be used long-leaf yellow pine; for S, 1500 lb per sq in, Table XIV, page 645; for a fib stress, S, of 1800 lb per sq in, Table XV, page 646; and for S equal to 1600 per sq in. Table XII, page 643, with the strength-values increased one-thi Of course, the sizes of the timbers are diminished as the assumed safe fib stresses are increased.

This example illustrates nearly all of the computations that are required determine the sizes of the joists and special beams or girders in any ordina floor-construction. The method of computation is the same for any flo load, the only difference being that the greater the live load assumed the grea will be the loads for which the beams must be proportioned. As will be se the most laborious computations are those for beams which receive loads fr different sources, and it will generally be found that the weakest portions any particular floor are the headers, trimmers and girders, and the beams wh support partitions.

The Strength of Mill-Floors. The beams and girders for mill-flo should be computed by the same general method illustrated in the forego examples, involving, (1) the determination of the loads on the beams girders and, (2) the sizes of the beams and girders required to support su loads.

Required Thickness of Plank Flooring. The thickness of the plank flo ing in mill construction may be determined by formulas (1) and (2):

$$\left.\begin{array}{r}\text{Thickness of plank in in}\\ \text{required for strength}\end{array}\right\} = \sqrt{\frac{\text{weight per sq ft} \times l^2}{24 \times A}}$$

$$\left.\begin{array}{r}\text{Thickness of plank in in}\\ \text{required for stiffness}\end{array}\right\} = \sqrt[3]{\frac{\text{weight per sq ft} \times l^3}{19.2 \times e_1}}$$

In these formulas, l is the span in feet, from center to center of beams, A constants for strength (page 628), and e_1 the constant for stiffness (page 664)

When the planks are connected by ¾-in splines, and extend over two sp Formula (1) may be used. If the planks are in single lengths from beam to be or are not splined, then Formula (2) should be used.

Tables IV to XI,* inclusive, show the safe loads for plank flooring of diffe woods, thicknesses and spans, derived from the formulas for strength and stiffn the values in the first horizontal line in the case of each thickness of pl

* Tables VIII to XI, inclusive, were calculated by Mr. F. E. Kidder and are reta from the preceding edition of the Pocket-Book. Tables IV to VII, inclusive, are a to conform to the most conservative fiber-stresses of the building codes and of the c chapters of the new edition. In the judgment of many constructors the higher valu Tables VIII to XI are safe when more favorable conditions of quality and dryne materials prevail. In using any of the tables, care must be taken to notice whe or not the safe loads given include the weight of the flooring itself. In the revisio this chapter the author is indebted to Professor F. H. Safford, of the University of P sylvania, for the computations required for the new Tables IV to VII and for the chec of Tables VIII to XI.

[illegible]ting the loads given by the formula for strength and the figures in the [illegible]ond line those given by the formula for stiffness. The span is supposed to be [illegible]sured from center to center of beams. The values given by the formula for [illegible]ngth should be considered safe only for splined floors and where the planks [illegible] continuous over at least two spans. If the thickness of the planks falls [illegible]t ¼ or even ⅛ in from the dimensions given, the safe loads must be mate-[illegible]ly reduced.

[illegible] Table IV, the modulus of elasticity, E, on which e_1 in the stiffness-formula [illegible]nds, is 1 500 000 lb per sq in, and the safe fiber-stress, S, on which the [illegible]tant for strength, A, depends, is 1 200 lb per sq in, A being 67. The safe [illegible]s given are within the requirements of all cities for strength and stiffness for [illegible]-leaf yellow pine, and of all cities for Douglas fir.

[illegible] Table V, E is 1 200 000 lb per sq in, S, 1 000 lb per sq in, and A is 56. [illegible] loads given satisfy the requirements of Chicago and of most other cities [illegible]strength for short-leaf yellow pine. The values given for stiffness, also, are [illegible]mmended for this wood.

[illegible] Table VI, E is 1 200 000 lb per sq in, S, 800 lb per sq in, and A is 44. [illegible] loads given satisfy the requirements of all cities for strength, for spruce, [illegible]ay pine and white pine, and the values given for stiffness, also, are [illegible]mmended for spruce. For Norway pine, $E = 1\ 100\ 000$ lb per sq in may [illegible]ed.

[illegible] Table VII, E is 1 000 000 lb per sq in, S, 700 lb per sq in, and A is 39. [illegible] loads given for strength can be used for any woods of that safe fiber-[illegible], and the loads for stiffness are recommended for white pine.

[illegible] Tables VIII, IX, X and XI, the safe loads are calculated from still other [illegible]s of S, A, E and e_1, indicated with each table, and may be used by those [illegible] wish to assume larger safe values for the strength and stiffness-factors [illegible]es where there are no restrictions from building laws. For any other values [illegible] or e_1 required, such values must be inserted in Formula (1) or (2) and the [illegible]nesses of the planks determined or the safe load determined for any given [illegible]ness of planks.

[illegible]te. It is to be noted that for ideal conditions and commercially dry lum-[illegible] protected from moisture, and when there is no impact, the given fiber-stresses [illegible]lexure may be increased from 30 to 40%. (See, also, important notes on [illegible]s 628, 637 and 647 regarding stresses in and loads on wooden beams.)

Table IV. Safe Live Loads* in Pounds per Square Foot for Plank Flo

See explanation on pages 730–1. The loads are based on the following values. Strength: S = 1 200 lb per sq in, A = 67; stiffness: E = 1 500 000 lb per sq in, e_1 =

LONG-LEAF YELLOW PINE AND DOUGLAS FIR †

Thickness of planks, in	Distance between centers of floor-beams, in feet							
	4	5	6	7	8	9	10	11
1⅞	353 229	226 117	157 68	115 43	88 29	70 20	....	....
2⅜	567 466	363 239	252 138	185 87	142 58	112 41	91 30	75 22
2¾	760 724	486 371	338 214	248 135	190 90	150 64	122 46	100 35
3½	....	788 764	547 442	402 278	308 187	243 131	197 95	163 72
4	....	....	715 660	525 416	402 278	318 196	257 143	213 107
5	....	....	....	820 812	628 544	496 382	402 278	332 209
6	....	....	....	....	904 940	715 660	579 481	478 361

* Weight of ceiling, if any, and also of the flooring itself is to be deducted from these v
† If S for Douglas fir is taken at 1000 lb per sq in, use Table V.

Table V. Safe Live Loads* in Pounds per Square Foot for Plank Floo

See explanation on pages 730–1. The loads are based on the following values. Strength: S = 1 000 lb per sq in, A = 56; stiffness: E = 1 200 000 lb per sq in, e_1

SHORT-LEAF YELLOW PINE

Thickness of planks, in	Distance between centers of floor-beams, in feet							
	4	5	6	7	8	9	10	11
1⅞	295 182	189 93	131 54	96 34	74 23	....	....	....
2⅜	474 370	303 189	211 110	155 69	118 46	94 32	76 24	....
2¾	635 574	406 294	282 170	207 107	159 72	125 50	102 37	84 28
3½	1 029	659 606	457 351	336 221	257 148	203 104	165 76	136 57
4	....	860	597 523	439 330	336 221	265 155	215 113	178 85
5	....	....	933	686 644	525 431	415 303	336 221	278 166
6	....	....	....	987	756 745	597 523	484 382	400 287

* Weight of ceiling, if any, and also of the flooring itself is to be deducted from values.

Table VI. Safe Live Loads* in Pounds per Square Foot for Plank Flooring

See explanation on pages 730–1. The loads are based on the following values. Strength: S = 800 lb per sq in, A = 44; stiffness: E = 1 200 000 lb per sq in, e_1 = 92

SPRUCE, NORWAY PINE AND WHITE PINE

Thickness of planks, in	Distance between centers of floor-beams, in feet								
	4	5	6	7	8	9	10	11	12
1⅞	232	148	103	76	58	.....	.....	.....	.....
	182	93	54	34	23	.....	.....	.....	.....
2⅜	372	238	165	122	93	74	60	.....	.....
	370	189	110	69	46	32	24	.....	.....
2¾	499	319	222	163	125	99	80	66	.....
	.....	294	170	107	72	50	37	28	.....
3½	809	517	359	264	202	160	129	107	89
	.....	.....	351	221	148	104	76	57	44
4	.....	676	469	345	264	209	169	140	117
	.....	.....	.....	330	221	155	113	85	65
5	.....	.....	733	539	412	326	264	218	183
	.....	.....	.....	.....	.....	303	221	166	128
6	.....	.....	.....	776	594	469	380	314	264
	.....	.....	.....	.....	.....	.....	.....	287	221

* Weight of ceiling, if any, and also of the flooring itself is to be deducted from these loads.

Table VII. Safe Live Loads* in Pounds per Square Foot for Plank Flooring

See explanation on pages 730–1. The loads are based on the following values. Strength: S = 700 lb per sq in, A = 39; stiffness: E = 1 000 000 lb per sq in, e_1 = 77

FOR HEMLOCK AND WOODS OF SIMILAR STRENGTH AND STIFFNESS

Thickness of planks, in	Distance between centers of floor-beams, in feet								
	4	5	6	7	8	9	10	11	12
1⅞	206	132	91	67	.....	.....	.....	.....	.....
	152	78	45	28	.....	.....	.....	.....	.....
2⅜	330	212	147	108	82	65	.....	.....	.....
	309	158	92	58	39	27	.....	.....	.....
2¾	442	283	197	146	111	87	71	.....	.....
	480	246	142	90	60	42	31	.....	.....
3½	717	459	319	234	179	142	115	95	80
	.....	.....	293	185	124	87	63	48	37
4	936	599	416	306	234	185	150	124	104
	.....	.....	.....	276	185	130	95	71	55
5	.....	936	650	478	366	289	234	193	163
	.....	.....	.....	.....	361	253	185	139	107
6	.....	.....	936	688	526	416	337	278	234
	.....	.....	.....	.....	.....	.....	319	240	185

* Weight of ceiling, if any, and also of the flooring itself is to be deducted from these loads.

Table VIII. Safe Live Loads* in Pounds per Square Foot for Plank Floo[ring]

See explanation on pages 730–1. The loads are based on the following values.
Strength: $S = 1\,800$ lb per sq in, $A = 100$; stiffness: $E = 1\,780\,000$ lb per sq in, $e_1 =$
Recommended by Mr. Kidder for

LONG-LEAF YELLOW PINE

Thickness of planks, in	Distance between centers of floor-beams, in feet								
	4	5	6	7	8	9	10	11	12
1⅞	515 258	325 126	222 68	160 38	120 21	92 11	72 5	....	... ...
2⅜	831 536	527 268	362 149	262 88	197 54	153 34	121 24	97 12	8
2¾	1 118 838	710 421	488 237	354 144	267 91	208 59	165 38	134 25	11 1
3½	....	1 158 884	798 504	582 310	442 202	345 136	276 94	225 67	18 4
4	....	....	1 046 759	763 470	580 308	454 210	364 148	296 106	24 7
5	....	....	....	1 200 934	913 618	716 427	576 304	471 223	39 16
6	....	....	....	....	1 322 1 081	1 038 751	836 540	686 398	5 3

* Weight of ceiling, if any, to be deducted. The weight of the flooring has been ded[ucted] from values derived from formulas. Deduction about 72 lb per cu ft floor-materia[l].

Table IX. Safe Live Loads* in Pounds per Square Foot for Plank Floorin[g]

See explanation on pages 730–1. The loads are based on the following values.
Strength: $S = 1\,620$ lb per sq in, $A = 90$; stiffness: $E = 1\,425\,000$ lb per sq in, $e_1 =$
Recommended by Mr. Kidder for

DOUGLAS FIR AND SHORT-LEAF YELLOW PINE

Thickness of planks, in	Distance center to center of floor-beams, in feet							
	4	5	6	7	8	9	10	11
1⅞	462 205	291 99	199 52	143 28	106 15	81 7	64	....
2⅜	747 428	473 212	324 117	234 68	176 41	136 25	107 14	....
2¾	1 005 670	637 335	438 187	317 112	239 69	185 44	147 28	119 17
3½	....	1 040 706	717 401	522 246	395 159	308 106	246 72	200 50
4	....	1 362 1 061	940 606	685 374	520 244	406 165	325 115	265 81
5	....	....	1 476 1 198	1 078 745	819 491	642 338	516 240	422 174
6	....	....	....	1 560 1 302	1 187 863	932 597	749 428	614 314

* Weight of ceiling, if any, to be deducted. The weight of the flooring has been de[ducted] from values derived from formulas. Deduction about 72 lb per cu ft floor-mater[ial].

Table X. Safe Live Loads* in Pounds per Square Foot for Plank Flooring

See explanation on pages 730–1. The loads are based on the following values.
Strength: S = 1 260 lb per sq in, A = 70; stiffness: E = 1 294 000 lb per sq in, e_1 = 100

Recommended by Mr. Kidder for

SPRUCE

Thickness of planks, in	Distance between centers of floor-beams, in feet								
	4	5	6	7	8	9	10	11	12
1⅞	360 188	227 92	155 49	111 28	83 15	64 8	50	....	....
2⅜	581 391	368 194	252 108	182 64	137 39	105 24	83 15	67	54 .....
2¾	782 612	496 307	341 173	247 104	186 66	144 42	115 28	93 18	76 .....
3½	1 228 1 274	781 644	548 367	391 225	296 146	231 98	184 68	150 47	124 33
4	....	1 060 968	731 554	533 343	405 225	317 153	253 108	207 77	171 56
5	....	....	1 148 1 093	839 682	638 450	500 311	402 212	329 162	273 120
6	....	....	....	1 213 1 188	924 789	725 548	583 394	478 290	400 220

* Weight of ceiling, if any, to be deducted. The weight of the flooring has been deducted from values derived from formulas. Deduction about 72 lb per cu ft floor-material.

Table XI. Safe Live Loads* in Pounds per Square Foot for Plank Flooring

See explanation on pages 730–1. The loads are based on the following values.
Strength: S = 1 080 lb per sq in, A = 60; stiffness: E = 1 073 000 lb per sq in, e_1 = 8

Recommended by Mr. Kidder for

WHITE PINE

Thickness of planks, in	Distance between centers of floor-beams, in feet								
	4	5	6	7	8	9	10	11	12
1⅞	307 153	193 74	131 39	94 21	70 11	53 5	41	....	....
2⅜	496 318	314 157	214 86	154 50	116 40	89 18	70 10	56	....
2¾	668 499	424 249	290 139	210 83	158 52	122 33	97 20	78 12	63 .....
3½	1 088 1 041	691 526	476 298	346 183	261 119	203 78	162 53	131 36	108 25
4	....	906 791	625 451	455 278	345 181	269 123	215 85	175 60	145 43
5	....	....	982 893	716 555	544 366	426 251	342 178	281 129	232 95
6	....	....	1 419 1 553	1 037 970	789 643	619 445	497 319	407 234	339 175

* Weight of ceiling, if any, to be deducted. The weight of the flooring has been deducted from values derived from formulas. Deduction about 72 lb per cu ft floor-material.

Tables for the Maximum Span of Floor-Joists. As the timbers commo used for floor-joists are sawed to regular sizes and are usually spaced eithe or 16 in on centers, it is practicable to show by means of tables the sizes of jo required to support given loads with given spans and spacings. Tables gi the MAXIMUM SAFE SPANS are the most convenient for general use, and the foll ing tables have accordingly been prepared. They show at a glance the m imum spans for which different sizes of floor-joists and ceiling-joists shoul used for different loads and spacings, and it is believed that they will be fo applicable to most buildings in which wooden floor-joists are used. By kno the size of a room and the purpose for which it is to be used, the sizes of floor-joists required can be determined at a glance. Incidentally the ta show, also, the kind of wood most economical to use. If, owing to the r being irregular in shape, the joists must be of different lengths, the spacin thickness of the joists may be varied, so that the same depth may be throughout.

Precautions Required in Using Tables. The precautions necessary in u these tables are in regard to the superimposed loads and the ACTUAL SIZE the timbers. The TOTAL LOADS for which the maximum spans have been puted are given at the head of each table. The actual weight of the (joists, flooring, plastering and deafening, if any) subtracted from the total will give the SUPERIMPOSED LOAD, that is, the load which the floor is expecte carry. If the ACTUAL SIZES of the joists are less than the NOMINAL DIMENSI the spans or spacings must be reduced from those given in the tables, an the STOCK SIZES of joists generally run from ¼ in to ⅜ in scant of the NOM DIMENSIONS, this fact should always be taken into account when determi upon the sizes of joists. In this connection it will be convenient to reme that 2-in joists, spaced 16 in on centers, have the same strength as 1½-in j 12 in on centers. A reduction should also be made for any CUTTING OF JOISTS that may be required. No allowance has been made for PARTITIONS when they are to be supported by the floor-joists, additional joists shou used or the span reduced according to the relative direction or position o partitions and joists.

Tables XII to XX. Tables XII to XVI, inclusive, were computed by FORMULA FOR STIFFNESS (Chapter XVI, page 636 and Chapter XVIII, pages on the assumption that the deflection should not exceed 1/40 in per foot of They are based on the values of E (the modulus of elasticity) recommende F. E. Kidder. Tables XVII to XX, inclusive, were computed by the FOR FOR STRENGTH (Chapter XVI, page 635), and values for S (the safe fiber-s recommended by Mr. Kidder. The spans given in Tables XII to XX, sive, come within the requirements of the Buffalo and Denver building and Tables XII, XIV, XV, XVI and XVII comply with the Chicago la very nearly with the New York law; but to comply with the Boston reduction of about one-sixth must be made from the spans given (1914).*

Tables XXI to XXIX † inclusive, were computed for reduced values of E modulus of elasticity,) S (the fiber-stress for flexure) and A (the constan flexural strength) in the formulas used, these values agreeing generally wi stresses throughout the revised handbook. Of these new tables, also, T XXI to XXV, inclusive, were computed by the FORMULA FOR STIFFNESS Tables XXVI to XXIX, inclusive, by the FORMULA FOR STRENGTH.

* Building Codes are frequently revised and must be consulted.

† In the revision of this chapter the author is indebted to Mr. A. T. North, M. A C. E., for valuable assistance in the computations required for the new Tables X XXIX.

Table XII. Maximum Span for Ceiling-Joists

See explanatory notes on page 736

Sizes of joists	Distance on centers	Hemlock, * E = 1 045 000		White pine, E = 1 073 000		Norway pine or spruce, E = 1 294 000		Douglas fir or Texas pine, E = 1 425 000		Long-leaf yellow pine, E = 1 780 000	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 20 pounds per square foot											
2X4	12	9	3	9	5	10	1	10	5	11	2
2X4	16	8	5	8	6	9	1	9	5	10	1
2X6	12	14	0	14	1	15	1	15	7	16	8
2X6	16	12	8	12	10	13	8	14	2	15	2
2X8	12	18	8	18	10	20	1	20	9	22	4
2X8	16	17	0	17	2	18	4	18	11	20	5
2X8	20	15	9	15	10	17	0	17	6	18	10
Total load, 24 pounds per square foot											
2X10	12	22	0	22	2	23	8	24	5	26	4
2X10	16	20	0	20	2	21	7	22	3	23	10
2X10	20	18	6	18	8	20	0	20	7	22	2
2X12	12	26	5	26	8	28	5	29	4	31	7
2X12	16	24	0	24	2	25	10	26	8	28	8
2X12	20	22	3	22	5	24	0	24	8	26	8

* E is the modulus of elasticity and is in pounds per square inch.

Table XIII. Maximum Span for Floor-Joists for Dwellings, Tenements and Grammar-School Rooms with Fixed Desks

See explanatory notes on page 736

Sizes of joists	Distance on centers	Hemlock, * E = 1 045 000		White pine, E = 1 073 000		Norway pine or spruce, E = 1 294 000		Douglas fir or Texas pine, E = 1 425 000		Long-leaf yellow pine, E = 1 780 000	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 60 pounds per square foot											
2X6	12	9	9	9	10	10	5	10	10	11	7
2X6	16	8	9	8	10	9	6	9	10	10	6
3X6	12	11	1	11	2	12	0	12	5	13	4
3X6	16	10	1	10	2	10	10	11	2	12	1
X8	12	12	11	13	1	13	11	14	5	15	6
X8	16	11	9	11	10	12	8	13	1	14	1
X8	12	14	9	14	11	16	0	16	6	17	8
X8	16	13	6	13	7	14	6	15	0	16	2
X10	12	16	2	16	4	17	5	18	0	19	4
X10	16	14	9	14	10	15	9	16	4	17	7
Total load, 66 pounds per square foot											
X10	12	18	0	18	1	19	3	20	0	21	6
X10	16	16	3	16	5	17	7	18	2	19	6
X12	12	18	10	19	0	20	3	20	10	22	6
X12	16	17	2	17	3	18	4	19	0	20	6
X12	12	21	6	21	8	23	2	24	0	25	9
X12	16	19	7	19	8	21	1	21	9	23	5
X14	12	22	0	22	2	23	8	24	4	26	3
X14	16	20	0	20	1	21	6	22	2	23	10
X14	12	23	8	23	10	25	6	26	3	28	3
X14	16	21	6	21	8	23	2	23	10	25	8
X14	12	25	4	25	4	27	1	28	0	30	1
X14	16	23	0	23	0	24	7	25	4	27	4

* E is the modulus of elasticity and is in pounds per square inch.

Table XIV. Maximum Span for Floor-Joists for Office-Buildings

See explanatory notes on page 736

Sizes of joists	Distance on centers	White pine, * $E=1\ 073\ 000$		Norway pine or spruce, $E=1\ 294\ 000$		Douglas fir or Texas pine, $E=1\ 425\ 000$		Long-le[illegible] yellow pi[illegible] $E=1\ 780$ [illegible]	
in	in	ft	in	ft	in	ft	in	ft	in
Total load, 93 pounds per square foot									
3×8	12	12	10	13	9	14	2	15	[illegible]
3×8	16	11	8	12	6	12	10	13	[illegible]
2×10	12	14	1	15	1	15	6	16	[illegible]
2×10	16	12	9	13	8	14	1	15	[illegible]
3×10	12	16	1	17	3	17	9	19	[illegible]
3×10	16	14	8	15	8	16	2	17	[illegible]
2×12	12	16	10	18	1	18	8	20	[illegible]
2×12	16	15	4	16	5	17	0	18	[illegible]
Total load, 96 pounds per square foot									
3 ×12	12	19	2	20	6	21	2	22	[illegible]
3 ×12	16	17	5	18	7	19	3	20	[illegible]
2 ×14	12	19	6	20	10	21	7	23	[illegible]
2 ×14	16	17	9	19	0	19	7	21	[illegible]
2½×14	12	21	1	22	6	23	2	25	[illegible]
2½×14	16	19	2	20	4	21	2	22	[illegible]
3 ×14	12	22	4	23	10	24	8	27	[illegible]
3 ×14	16	20	4	21	8	22	5	24	[illegible]

* E is the modulus of elasticity and is in pounds per square inch.

Table XV. Maximum Span for Floor-Joists for Churches and Theaters with Fixed Seats

See explanatory notes on page 736

Sizes of joists	Distance on centers	White pine, * $E=1\ 073\ 000$		Norway pine or spruce, $E=1\ 294\ 000$		Douglas fir or Texas pine, $E=1\ 425\ 000$		Long-le[illegible] yellow pi[illegible] $E=1\ 780$ [illegible]	
in	in	ft	in	ft	in	ft	in	ft	in
Total load, 102 pounds per square foot									
3×8	12	12	6	13	4	13	9	14	[illegible]
3×8	16	11	4	12	2	12	6	13	[illegible]
2×10	12	13	7	14	7	15	1	16	[illegible]
2×10	16	12	4	13	3	13	8	14	[illegible]
3×10	12	15	8	16	9	17	3	18	[illegible]
3×10	16	14	2	15	2	15	8	16	[illegible]
2×12	12	16	5	17	7	18	1	19	[illegible]
2×12	16	14	10	15	11	16	5	17	[illegible]
Total load, 105 pounds per square foot									
3 ×12	12	18	7	19	11	20	6	22	[illegible]
3 ×12	16	16	10	18	1	18	7	20	[illegible]
2 ×14	12	19	0	20	3	20	10	22	[illegible]
2 ×14	16	17	3	18	5	19	0	20	[illegible]
2½×14	12	20	4	21	9	22	6	24	[illegible]
2½×14	16	18	7	19	10	20	6	22	[illegible]
3 ×14	12	21	8	23	2	23	10	25	[illegible]
3 ×14	16	19	8	21	1	21	9	23	[illegible]

* E is the modulus of elasticity and is in pounds per square inch.

Table XVI. Maximum Span for Floor-Joists for Assembly-Halls and Corridors

See explanatory notes on page 736

Sizes of joists in	Distance on centers in	White pine, * $E = 1\ 073\ 000$ ft	in	Norway pine or spruce, $E = 1\ 294\ 000$ ft	in	Douglas fir or Texas pine, $E = 1\ 425\ 000$ ft	in	Long-leaf yellow pine, $E = 1\ 780\ 000$ ft	in
Total load, 123 pounds per square foot									
3X8	12	11	7	12	7	13	0	14	0
3X8	16	10	8	11	4	11	9	12	8
2X10	12	12	10	13	9	14	2	15	2
2X10	16	11	7	12	6	12	10	13	10
3X10	12	14	8	15	8	16	2	17	5
3X10	16	13	4	14	3	14	9	15	10
2X12	12	15	4	16	6	17	0	18	3
2X12	16	14	0	15	0	15	5	16	7
Total load, 126 pounds per square foot									
3 X12	12	17	6	18	8	19	3	20	9
3 X12	16	15	10	17	0	17	7	18	11
2 X14	12	17	10	19	1	19	8	21	2
2 X14	16	16	2	17	4	17	11	19	3
2½X14	12	19	3	20	6	21	2	22	9
2½X14	16	17	6	18	8	19	3	20	9
3 X14	12	20	5	21	9	22	6	24	3
3 X14	16	18	7	19	10	20	6	22	1

* E is the modulus of elasticity and is in pounds per square inch.

Table XVII. Maximum Span for Floor-Joists for Retail Stores

See explanatory notes on page 736

Sizes of joists in	Distance on centers in	White pine, $S = 1\ 080$ lb per sq in * $A = 60$ ft	in	Norway pine or spruce, $S = 1\ 260$ lb per sq in $A = 70$ ft	in	Douglas fir or Texas pine, $S = 1\ 620$ lb per sq in $A = 90$ ft	in	Long-leaf yellow pine, $S = 1\ 800$ lb per sq in $A = 100$ ft	in
Total load, 174 pounds per square foot									
3X8	12	11	6	12	5	14	1	14	9
3X8	16	9	11	10	2	12	2	12	9
2X10	12	11	8	12	8	14	5	15	1
2X10	16	10	2	10	11	12	5	13	1
3X10	12	14	4	15	6	17	7	18	7
3X10	16	12	5	13	5	15	2	16	0
2X12	12	14	1	15	2	17	2	18	2
2X12	16	12	2	13	1	14	11	15	8
Total load, 177 pounds per square foot									
3 X12	12	17	2	18	5	20	11	22	1
3 X12	16	14	10	16	0	18	2	19	1
2 X14	12	16	3	17	7	19	11	21	1
2 X14	16	14	2	15	2	17	3	18	2
2½X14	12	18	2	19	7	22	3	23	6
2½X14	16	15	9	17	0	19	3	20	4
3 X14	12	19	11	21	6	24	5	25	8
3 X14	16	17	3	18	7	21	2	22	3

* A in the tables is the coefficient in formulas for beams and is one-eighteenth of th[e] [assu]med flexural fiber-stress, S.

Table XVIII.* Maximum Span for Rafters. Shingled Roofs not Plastere

See explanatory notes on page 736

Total load, 48 pounds per square foot

Sizes of joists	Distance on centers	Hemlock, $S=990$ lb per sq in † $A=55$		White pine, $S=1\,080$ lb per sq in $A=60$		Norway pine or spruce, $S=1\,260$ lb per sq in $A=70$		Douglas fir or Texas pine, $S=1\,620$ lb per sq in $A=90$		Long-lea yellow pi $S=1\,80$ lb per sq $A=100$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
2×4	16	7	4	7	9	8	4	9	6	10	[illegible]
2×4	20	6	7	6	10	7	6	8	6	8	[illegible]
2×6	16	11	1	11	7	12	6	14	2	15	[illegible]
2×6	20	9	11	10	4	11	2	12	8	13	[illegible]
3×6	16	13	7	14	2	15	3	17	5	18	[illegible]
3×6	20	12	2	12	8	13	8	15	7	16	[illegible]
2×8	16	14	9	15	6	16	8	18	11	20	[illegible]
2×8	20	13	3	13	10	14	11	16	11	17	[illegible]
2×8	24	12	1	12	7	13	7	15	6	16	[illegible]
2×10	16	18	6	19	3	20	10	23	8	25	[illegible]
2×10	20	16	7	17	3	18	8	21	2	22	[illegible]
2×10	24	15	1	15	9	17	0	19	3	20	[illegible]

Table XIX.* Maximum Span for Rafters. Slate Roofs not Plastered, or Shingle Roofs Plastered

See explanatory notes on page 736

Total load, 57 pounds per square foot

Sizes of joists	Distance on centers	Hemlock, $S=990$ lb per sq in † $A=55$		White pine, $S=1\,080$ lb per sq in $A=60$		Norway pine or spruce, $S=1\,260$ lb per sq in $A=70$		Douglas fir or Texas pine, $S=1\,620$ lb per sq in $A=90$		Long-le yellow pi $S=1\,80$ lb per sq $A=100$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
2×4	16	6	9	7	1	7	7	8	8	9	[illegible]
2×4	20	6	0	6	4	6	9	7	9	8	[illegible]
2×6	16	10	2	10	7	11	6	13	0	13	[illegible]
2×6	20	9	1	9	6	10	2	11	7	12	[illegible]
3×6	16	12	6	13	0	14	1	15	11	16	[illegible]
3×6	20	11	1	11	8	12	7	14	3	15	[illegible]
2×8	16	13	7	14	2	15	3	17	4	18	[illegible]
2×8	20	12	2	12	8	13	8	15	6	16	[illegible]
2×8	24	11	1	11	7	12	6	14	2	14	[illegible]
3×8	16	16	7	17	4	18	9	21	3	22	[illegible]
3×8	20	14	10	15	6	16	9	19	0	20	[illegible]
3×8	24	13	7	14	2	15	3	17	4	18	[illegible]
2×10	16	17	0	17	8	19	2	21	7	22	[illegible]
2×10	20	15	2	15	10	17	1	19	4	20	[illegible]
2×10	24	13	10	14	6	15	7	17	8	18	[illegible]

* Tables XVIII, XIX and XX are intended for climates where a 2-ft snow-fall may expected. In the Southern States, where there is very little snow, the spans in T XVIII will be safe for slate or gravel roofs if the joists are sawed to the full dimensi Variations in "Safe spans" in different tables, for the same kind of wood, depend u the assumed safe flexural fiber-stress or modulus of elasticity or both.

† A in the tables is the coefficient in formulas for beams and is one-eighteenth of assumed flexural fiber-stress, S.

Table XX.* Maximum Span for Rafters. Slate Roofs Plastered, or Gravel Roofs not Plastered

See explanatory notes on page 736

Total load, 66 pounds per square foot											
Sizes of joists	Distance on centers	Hemlock, $S=990$ lb per sq in † $A=55$		White pine, $S=1\,080$ lb per sq in $A=60$		Norway pine or spruce, $S=1\,260$ lb per sq in $A=70$		Douglas fir or Texas pine, $S=1\,620$ lb per sq in $A=90$		Long-leaf yellow pine, $S=1\,800$ lb per sq in $A=100$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
2X6	16	9	5	9	10	10	8	12	1	12	9
2X6	20	8	6	8	10	9	6	10	9	11	5
3X6	16	11	7	12	1	13	1	14	10	15	7
3X6	20	10	4	10	10	11	8	13	3	14	0
2X8	16	12	7	13	2	14	2	16	2	17	0
2X8	20	11	3	11	9	12	9	14	5	15	2
2X8	24	10	3	10	9	11	7	13	2	13	10
3X8	16	15	5	16	1	17	5	19	9	20	10
3X8	20	13	9	14	5	15	3	17	8	18	8
3X8	24	12	7	13	2	14	2	16	2	17	0
2X10	16	15	9	16	6	17	9	20	2	21	3
2X10	20	14	1	14	8	15	11	18	0	19	0
2X10	24	12	10	13	5	14	6	16	6	17	5
2X12	16	18	10	19	9	21	4	24	2	25	6
2X12	20	16	10	17	8	19	1	21	8	22	10
2X12	24	15	5	16	1	17	5	19	9	20	10

* Tables XVIII, XIX and XX are intended for climates where a 2-ft snow-fall may be pected. In the Southern States, where there is very little snow, the spans in Table III will be safe for slate or gravel roofs if the joists are sawed to the full dimensions. riations in "Safe spans" in different tables, for the same kind of wood, depend upon e assumed safe flexural fiber-stress or modulus of elasticity or both.

† A in the tables is the coefficient in formulas for beams and is one-eighteenth of the umed flexural fiber-stress, S.

Table XXI. Maximum Span for Ceiling-Joists

See explanatory notes on page 736

Sizes of joists	Distance on centers	Hemlock, * $E = 900\ 000$		White pine, $E = 1\ 000\ 000$		Norway pine, $E = 1\ 100\ 000$		Short-leaf yellow pine, spruce, $E = 1\ 200\ 000$		Long-leaf yellow pine, Douglas fir, $E = 1\ 500\ 000$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 20 pounds per square foot											
2×4	12	8	11	9	3	9	6	9	10	10	7
2×4	16	8	1	8	5	8	8	8	11	9	7
2×6	12	13	5	13	10	14	4	14	9	15	10
2×6	16	12	2	12	7	13	0	13	5	14	5
2×8	12	17	10	18	6	19	1	19	8	21	2
2×8	16	16	3	16	10	17	4	17	10	19	3
2×8	20	15	1	15	7	16	1	16	7	17	10
Total load, 24 pounds per square foot											
2×10	12	21	0	21	9	22	5	23	1	24	11
2×10	16	19	1	19	8	20	5	21	0	22	2
2×10	20	17	8	18	4	18	11	19	6	21	0
2×12	12	25	2	26	0	26	11	27	9	29	11
2×12	16	22	11	23	9	24	6	25	2	27	2
2×12	20	21	3	22	0	22	9	23	5	25	2

* E is the modulus of elasticity and is in pounds per square inch.

Table XXII. Maximum Span for Floor-Joists for Dwellings, Tenements and Grammar-School Rooms with Fixed Desks

See explanatory notes on page 736

Sizes of joists	Distance on centers	Hemlock, * $E = 900\ 000$		White pine, $E = 1\ 000\ 000$		Norway pine, $E = 1\ 100\ 000$		Short-leaf yellow pine, spruce, $E = 1\ 200\ 000$		Long-leaf yellow pine, Douglas fir, $E = 1\ 500\ 000$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 60 pounds per square foot											
2× 6	12	9	3	9	7	9	11	10	3	11	0
2× 6	16	8	5	8	9	9	0	9	3	10	0
3× 6	12	10	8	11	0	11	4	11	8	12	7
3× 6	16	9	8	10	0	10	4	10	8	11	5
2× 8	12	12	4	12	10	12	3	13	8	14	8
2× 8	16	11	3	11	8	12	0	12	4	13	4
3× 8	12	14	2	14	8	15	2	15	7	16	10
3× 8	16	12	11	13	4	13	9	14	2	15	3
2×10	12	15	6	16	0	16	7	17	0	18	3
2×10	16	14	1	14	7	15	0	15	6	16	[illegible]
Total load, 66 pounds per square foot											
3 ×10	12	17	2	17	9	18	4	18	11	20	4
3 ×10	16	15	7	16	2	16	8	17	2	18	[illegible]
2 ×12	12	18	0	18	8	19	3	19	8	21	4
2 ×12	16	16	4	16	11	17	8	18	0	19	[illegible]
3 ×12	12	20	7	21	4	22	0	22	8	24	[illegible]
3 ×12	16	18	8	19	4	20	0	20	7	22	[illegible]
2 ×14	12	21	0	21	11	22	5	23	1	24	[illegible]
2 ×14	16	19	1	19	9	20	5	21	0	22	[illegible]
2½×14	12	22	7	23	5	24	2	24	11	26	[illegible]
2½×14	16	20	6	21	3	21	11	22	7	24	[illegible]
3 ×14	12	24	0	24	10	25	8	26	5	28	[illegible]
3 ×14	16	21	10	22	7	23	4	24	0	25	[illegible]

* E is the modulus of elasticity and is in pounds per square inch.

Table XXIII. Maximum Span for Floor-Joists for Office-Buildings

See explanatory notes on page 736

Sizes of joists	Distance on centers	Hemlock, * $E=900\ 000$		White pine, $E=1\ 000\ 000$		Norway pine, $E=1\ 100\ 000$		Short-leaf yellow pine, spruce, $E=1\ 200\ 000$		Long-leaf yellow pine, Douglas fir, $E=1\ 500\ 000$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 93 pounds per square foot											
3X 8	12	12	3	12	8	13	1	13	6	14	6
3X 8	16	11	1	11	6	11	11	12	3	13	2
2X10	12	13	4	13	10	14	3	14	8	15	10
2X10	16	12	2	12	7	13	0	13	4	14	5
3X10	12	15	4	15	10	16	4	16	10	18	2
3X10	16	13	11	14	5	14	10	15	4	16	7
2X12	12	16	0	16	7	17	2	17	8	19	0
2X12	16	14	7	15	1	15	7	16	0	17	3
Total load, 96 pounds per square foot											
2 X12	12	18	2	18	10	19	5	20	0	21	6
2 X12	16	16	6	17	1	17	8	18	2	19	7
2 X14	12	18	6	19	2	19	10	20	5	21	11
2 X14	16	16	10	17	5	18	0	18	6	19	11
2½X14	12	19	11	20	8	21	4	22	0	23	8
2½X14	16	18	2	18	9	19	5	19	11	21	6
3 X14	12	21	2	21	11	22	8	23	4	25	2
3 X14	16	19	3	19	11	20	7	21	2	22	10

* E is the modulus of elasticity and is in pounds per square inch.

Table XXIV. Maximum Span for Floor-Joists for Churches and Theaters with Fixed Seats

See explanatory notes on page 736

Sizes of joists	Distance on centers	Hemlock, * $E=900\ 000$		White pine, $E=1\ 000\ 000$		Norway pine, $E=1\ 100\ 000$		Short-leaf yellow pine, spruce, $E=1\ 200\ 000$		Long-leaf yellow pine, Douglas fir, $E=1\ 500\ 000$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 102 pounds per square foot											
3X 8	12	11	10	12	3	12	8	13	1	14	1
3X 8	16	10	9	11	2	11	6	11	11	12	9
2X10	12	12	11	13	5	13	10	14	3	15	4
2X10	16	11	9	12	2	12	7	13	0	13	11
3X10	12	14	10	15	4	15	10	16	4	17	7
3X10	16	13	6	13	11	14	5	14	10	16	0
2X12	12	15	7	16	1	16	8	17	1	18	5
2X12	16	14	2	14	8	15	1	15	7	16	9
Total load, 105 pounds per square foot											
2 X12	12	17	8	18	3	18	10	19	5	20	11
2 X12	16	16	0	16	7	17	1	17	8	19	0
2 X14	12	18	0	18	7	19	3	19	8	21	4
2 X14	16	16	4	16	11	17	5	18	0	19	4
2½X14	12	19	4	20	1	20	8	21	4	23	0
2½X14	16	17	7	18	3	18	10	19	4	20	11
3 X14	12	20	7	21	4	22	0	22	8	24	5
3 X14	16	18	8	19	4	20	0	20	7	22	2

* E is the modulus of elasticity and is in pounds per square inch.

Table XXV. Maximum Span for Floor-Joists for Assembly-Halls and Corrid

See explanatory notes on page 736

Sizes of joists	Distance on centers	Hemlock, * $E=900\ 000$		White pine, $E=1\ 000\ 000$		Norway pine, $E=1\ 100\ 000$		Short-leaf yellow pine, spruce, $E=1\ 200\ 000$		Long-lea yellow pi Douglas f $E=1\ 500$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 123 pounds per square foot											
3×8	12	11	2	11	7	11	11	12	3	13	[illegible]
3×8	16	11	0	10	6	10	10	11	2	12	[illegible]
2×10	12	12	2	12	7	13	0	13	5	14	[illegible]
2×10	16	11	1	11	5	11	10	12	2	13	[illegible]
3×10	12	13	11	14	5	14	11	15	4	16	[illegible]
3×10	16	12	8	13	1	13	7	13	11	15	[illegible]
2×12	12	14	7	15	2	15	8	16	1	17	[illegible]
2×12	16	13	3	13	9	14	2	14	8	15	[illegible]
Total load, 126 pounds per square foot											
3 ×12	12	16	7	17	2	17	9	18	3	19	[illegible]
3 ×12	16	15	1	15	7	16	1	16	7	17	[illegible]
2 ×14	12	16	11	17	6	18	1	18	7	20	[illegible]
2 ×14	16	15	4	15	11	16	5	16	11	18	[illegible]
2½×14	12	18	2	18	10	19	6	20	1	21	[illegible]
2½×14	16	16	8	17	3	17	10	18	4	19	[illegible]
3 ×14	12	19	4	20	1	20	8	21	4	22	[illegible]
3 ×14	16	17	7	18	3	18	10	19	4	20	[illegible]

* E is the modulus of elasticity and is in pounds per square inch.

Table XXVI. Maximum Span for Floor-Joists for Retail Stores

See explanatory notes on page 736

Sizes of joists	Distance on centers	Hemlock, $S=600$ lb per sq in * $A=33⅓$		White pine, spruce, $S=700$ lb per sq in $A=38.88$		Norway pine, $S=800$ lb per sq in $A=44.44$		Douglas fir, short-leaf yellow pine, $S=1\ 000$ lb per sq in $A=55.55$		Souther long-le yellow pi $S=1\ 200$ per sq i $A=66⅔$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
Total load, 174 pounds per square foot											
3×8	12	8	7	9	3	9	11	11	1	12	[illegible]
3×8	16	7	5	8	0	8	7	9	7	10	[illegible]
2×10	12	8	9	9	5	10	1	11	4	12	[illegible]
2×10	16	7	7	8	2	8	9	9	10	10	[illegible]
3×10	12	10	9	11	7	12	5	13	10	15	[illegible]
3×10	16	9	3	10	0	10	9	12	0	13	[illegible]
2×12	12	10	6	11	4	12	2	13	7	14	[illegible]
2×12	16	9	1	9	10	10	6	11	9	12	[illegible]
Total load, 177 pounds per square foot											
3 ×12	12	12	6	13	6	14	6	16	2	17	[illegible]
3 ×12	16	10	10	11	9	12	6	14	0	15	[illegible]
2 ×14	12	12	2	13	1	14	0	15	8	17	[illegible]
2 ×14	16	10	6	11	4	12	2	13	7	14	[illegible]
2½×14	12	13	7	14	8	15	8	17	6	19	[illegible]
2½×14	16	11	9	12	8	13	7	15	2	16	[illegible]
3 ×14	12	16	8	18	0	19	3	21	6	23	[illegible]
3 ×14	16	14	5	15	7	16	8	18	8	20	[illegible]

* A in the tables is the coefficient in formulas for beams and is one-eighteenth of allowable flexural fiber-stress, S. For values of A for other woods, see Table II, 628.

ble XXVII.* Maximum Span for Rafters. Shingled Roofs, not Plastere

See explanatory notes on page 736

Total load, 48 pounds per square foot

Sizes of joists	Distance on centers	Hemlock, $S=600$ lb per sq in † $A=33\frac{1}{3}$		White pine, spruce, $S=700$ lb per sq in $A=38.88$		Norway pine, $S=800$ lb per sq in $A=44.44$		Douglas fir, short-leaf yellow pine, $S=1\,000$ lb per sq in $A=55.55$		Southern long-leaf yellow pin $S=1\,200$ l per sq in $A=66\frac{2}{3}$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
2X 4	16	5	9	6	3	6	8	7	5	8	2
2X 4	20	5	2	5	7	5	11	6	8	7	4
2X 6	16	8	8	9	4	10	0	11	2	12	3
2X 6	20	7	9	8	4	8	11	10	0	10	11
3X 6	16	10	7	11	5	12	3	13	8	15	0
3X 6	20	9	6	10	3	10	11	12	3	13	5
2X 8	16	11	6	12	6	13	4	14	11	16	4
2X 8	20	10	4	11	2	11	11	13	4	14	7
2X 8	24	9	5	10	2	10	11	12	2	13	4
2X10	16	14	5	15	7	16	8	18	8	20	5
2X10	20	12	11	13	11	14	11	16	8	18	3
2X10	24	11	9	12	9	13	7	15	2	16	8

Table XXVIII.* Maximum Span for Rafters. Slate Roofs, not Plastered, or Shingled Roofs, Plastered

See explanatory notes on page 736

Total load, 57 pounds per square foot

Sizes of joists	Distance on centers	Hemlock, $S=600$ lb per sq in † $A=33\frac{1}{3}$		White pine, spruce, $S=700$ lb per sq in $A=38.88$		Norway pine, $S=800$ lb per sq in $A=44.44$		Douglas fir, short leaf yellow pine $S=1\,000$ lb per sq in $A=55.55$		Southern long-leaf yellow pin $S=1\,200$ l per sq in $A=66\frac{2}{3}$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
2X 4	16	5	3	5	9	6	1	6	10	7	6
2X 4	20	4	9	5	1	5	6	6	1	6	8
2X 6	16	7	11	8	7	9	2	10	3	11	3
2X 6	20	7	1	7	8	8	2	9	2	10	1
3X 6	16	9	9	10	6	11	3	12	7	13	9
3X 6	20	8	8	9	5	10	1	11	3	12	4
2X 8	16	10	7	11	5	12	3	13	8	15	0
2X 8	20	9	6	10	3	10	11	12	3	13	5
2X 8	24	8	8	9	4	10	0	11	2	12	3
3X 8	16	13	0	14	0	15	0	16	9	18	4
3X 8	20	11	7	12	6	13	5	15	0	16	4
3X 8	24	10	7	11	5	12	3	13	8	15	0
2X10	16	13	3	14	4	15	3	17	1	18	9
2X10	20	11	10	12	9	13	8	15	3	16	9
2X10	24	10	10	11	8	12	6	13	11	15	3

* Tables XXVII, XXVIII and XXIX are intended for climates where a 2-ft snow-f ay be expected. In the Southern States, where there is very little snow, the spans ble XXVII will be safe for slate or gravel roofs if the joists are sawed to the full dime s. Variations in "Safe spans" in different tables, for the same kind of wood, depe n the assumed safe flexural fiber-stress or modulus of elasticity or both.

† See foot-note with Table XXVI.

Table XXIX.* Maximum Span for Rafters. Slate Roofs, Plastered, and Gravel Roofs, not Plastered

See explanatory notes on page 736

Total load, 66 pounds per square foot

Sizes of joists	Distance on centers	Hemlock, $S=600$ lb per sq in † $A=33\frac{1}{3}$		White pine, spruce, $S=700$ lb per sq in $A=38.88$		Norway pine, $S=800$ lb per sq in $A=44.44$		Douglas fir, short-leaf yellow pine, $S=1\,000$ lb per sq in $A=55.55$		Souther long-lea yellow pi $S=1\,200$ per sq in $A=66\frac{2}{3}$	
in	in	ft	in	ft	in	ft	in	ft	in	ft	in
2×6	16	7	5	8	0	8	6	9	6	10	[illegible]
2×6	20	6	7	7	2	7	7	8	6	9	[illegible]
3×6	16	9	0	9	9	10	5	11	8	12	[illegible]
3×6	20	8	1	8	9	9	4	10	5	11	[illegible]
2×8	16	9	10	10	8	11	4	12	8	13	[illegible]
2×8	20	8	10	9	6	10	2	11	4	12	[illegible]
2×8	24	8	0	8	8	9	3	10	5	11	[illegible]
3×8	16	12	1	13	0	13	11	15	7	17	[illegible]
3×8	20	10	9	11	8	12	5	13	11	15	[illegible]
3×8	24	9	10	10	8	11	4	12	8	13	[illegible]
2×10	16	12	4	13	3	14	2	15	11	17	[illegible]
2×10	20	11	0	11	11	12	9	14	2	15	[illegible]
2×10	24	10	1	10	10	11	10	13	0	14	[illegible]
2×12	16	14	9	15	11	17	1	19	1	20	[illegible]
2×12	20	13	2	14	3	15	3	17	1	18	[illegible]
2×12	24	12	1	13	0	13	11	15	7	17	[illegible]

* Tables XXVII, XXVIII and XXIX are intended for climates where a 2-ft snow-may be expected. In the Southern States, where there is very little snow, the spans Table XXVII will be safe for slate or gravel roofs if the joists are sawed to the full dim sions. Variations in "Safe spans" in different tables, for the same kind of wood, dep upon the assumed safe flexural fiber-stress or modulus of elasticity or both.

† See foot-note with Table XXVI.

To Determine the Strength of an Existing Floor. When a buildi is leased for mercantile or manufacturing purposes the tenant will genera desire to know the greatest load which it will be safe to put upon the floo and some building laws require that the safe load for the floors in certain clas of buildings shall be computed and posted in a conspicuous place in each sto It is therefore important that every architect should know how to compute safe strength of any existing floor. The problem is practically the reverse that of proportioning a floor to a given load. In speaking of the strength o floor a distinction should be made between the safe strength and the safe lo The SAFE STRENGTH should mean the maximum safe load for the beams, inclu ing the weight of the construction, flooring and ceiling, while the SAFE LO refers to the maximum load which may safely be placed upon the floor. T safe load is found by first computing the safe strength and then subtracti the weight of the materials forming the floor, including the ceiling below, if th is one. The most convenient measurement for either the SAFE STRENGTH the SAFE LOAD of a floor is in pounds per square foot. The following examp will serve to show the method of determining the safe load for an ordinary wa house-floor.

Example 4. It is required to determine the safe load per square foot fo floor framed as shown in Fig. 4, the building being in a city the laws of wh

allow 1 200 lb per sq in for the safe flexure fiber-stress for the wood of which the joists and girders are made. The joists are covered with two thicknesses of ⅞-in flooring and the ceiling below is corrugated iron.

Solution. The first step will be to find the SAFE STRENGTH of the 22-ft joists. As this is a warehouse-floor we will use the tables for strength throughout. From Table XII, page 643, for $S = 1\ 200$ lb per sq in, we find the safe strength of a 1 by 14-in joist of 22-ft span to be 1 188 lb; hence the strength of a 2½ by 14-in joist will be $1\ 188 \times 2½ = 2\ 970$ lb. As the joists are 16 in on centers, each joist supports a floor-area of $1⅓ \times 22$ ft $= 29⅓$ sq ft. The SAFE STRENGTH PER SQUARE FOOT of this portion of the floor will therefore

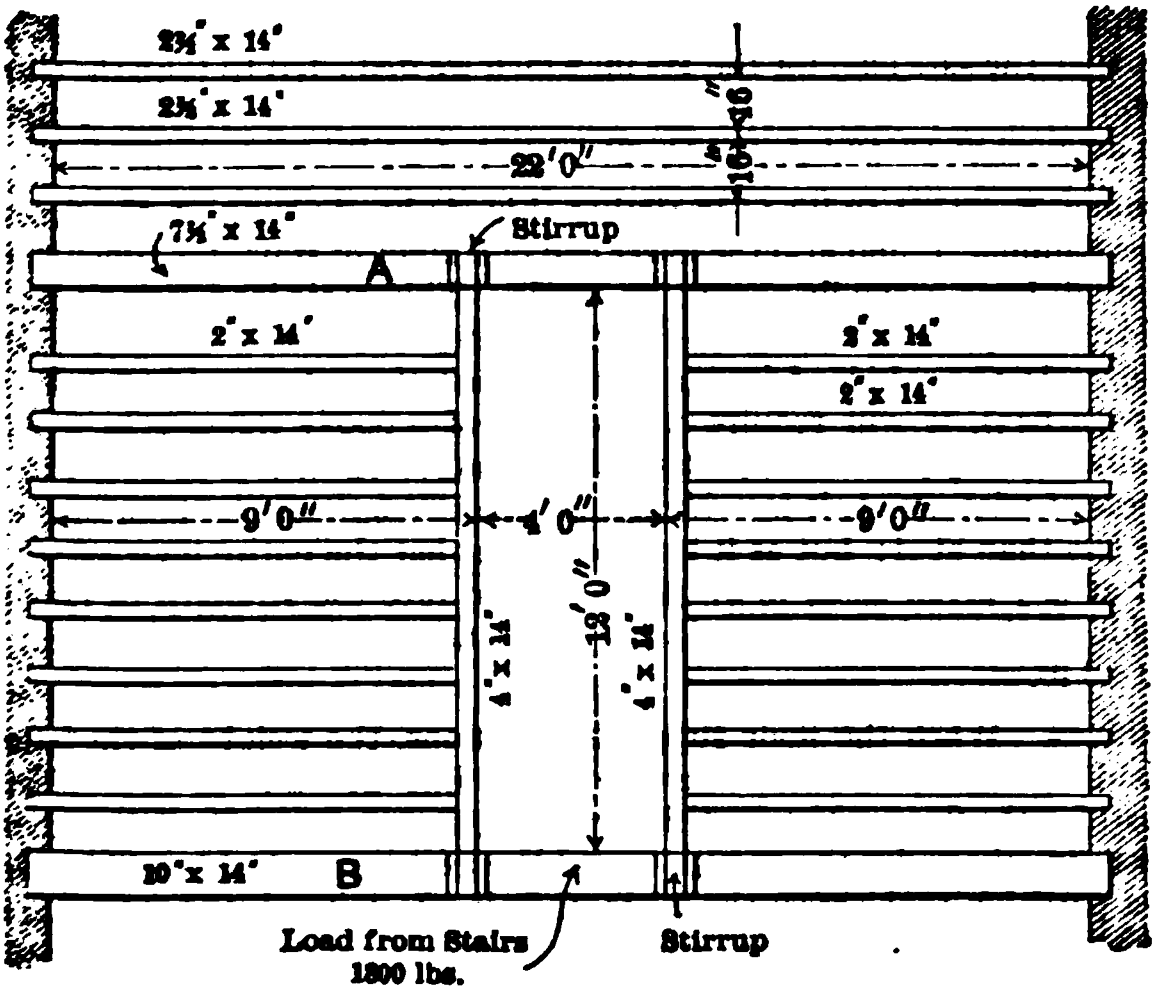


Fig. 4. Plan of a Warehouse-floor

be $2\ 970/29.3 = 101$ lb. Suppose the estimated weight of the floor per square foot is 8 lb for the joists, 6 lb for the flooring and 1 lb for the corrugated-iron ceiling, or, say, 15 lb in all. Then the SAFE LOAD PER SQUARE FOOT for the 22-ft joists will be $101 - 15 = 86$ lb.

The Headers. We will next find the safe load for the 4 by 14-in headers at each side of the stair-well. As the tail-beams are framed into the headers, we should deduct one inch from the thickness of each header for the loss of strength in framing, leaving 3 by 14 in for the effective dimension of each. From Table XII, page 643, we find the safe strength of a 1 by 14-in beam of 12-ft span to be 1 867 lb. Hence the strength of the 3 by 14 will be $1\ 867 \times 3 = 5\ 601$ lb. The floor-area supported by each header is $4½ \times 12$ ft $= 54$ sq ft; hence the SAFE STRENGTH of the header per square foot of floor is $5\ 601/54 = 104$ lb. Deducting the weight of the floor per square foot, we have $104 - 15 = 89$ lb for the SAFE LOAD.

Trimmer A. Trimmer A (Fig. 4) supports about the same amount of floor-area as one of the common joists, and supports, also, the ends of the headers. Deducting 2½ in, the thickness of the common joists, we have a 5 by 14-in beam

left to support the headers. As the headers are supported in iron stirrups, beam-hangers, no deduction in strength need be made for framing. To find t safe strength of a beam loaded with two concentrated loads, equidistant fr the supports, we must use Formula (14), Fig. 11, page 631. In this case a 8 ft 10 in, or $8\frac{5}{6}$ ft and $A = 1\,200/18 = 66.7$ (Table XII, page 643).

Applying the formula, the safe load at each joint $= 5 \times 14 \times 14 \times 66.7/4$ $8\frac{5}{6} = 1\,848$ lb.

The floor-area supported by one stirrup is equal to one-half of the area su ported by the header, or 27 sq ft; hence the safe strength per square foot the 5 by 14-in header is $1\,848/27 = 68$ lb, and deducting 15 lb per sq ft the weight of the floor, we have 53 lb per sq ft as the safe load that the trimm will support on the floor at each side of the stairs. Considering, as fou above, that the safe load for the $2\frac{1}{2}$ in, which we deducted to take the place o common joist, is 86 lb per sq ft, we might consider the safe load for the trimm as the average of 86 and 53, or about 70 lb per sq ft.

Trimmer B. This 10 by 14-in timber (Fig. 4) has to support the same flo loads as trimmer A, and also the lower end of a flight of stairs for which an allo ance of at least 1 800 lb should be made. This stair-load being practica concentrated at the middle of the trimmer is equivalent to a distributed load 3 600 lb. As the safe load for a 1 by 14-in joist of 22-ft span is 1 188 lb (Tal XII, page 643), it will require a thickness of $3\,600/1\,188 = 3$ in to supp the stairs, leaving 7 in to support the floor-loads. As this is $\frac{1}{2}$ in less than t thickness of trimmer A, it is evident that the strength of the floor at B will a little less than at A; but as it is improbable that the entire floor-space will loaded at any given time, it would be safe to rate the strength of the floor each side of the stairway at 70 lb per sq ft, LIVE LOAD, and beyond the sta way at 86 lb.

Partitions. When the floor supports partitions, the weight of the latter a any load resting upon them must be taken into account in determining t safe load for the floor. If a partition runs the same way as the joists, then o the joist directly under the partition, and the joists at each side will be affect but if a partition runs across the joists, then it affects the safe load of the ent floor.

Example 5. Suppose that the 22-ft joists in the floor shown in Fig. 4 have support a plastered partition 12 ft high, running across the joists half-way b tween the walls. What will be the safe load for the floor?

Solution. A plastered partition with 2 by 4 or 2 by 6-in studs, set 16 in centers, weighs about 20 lb per sq ft of partition-face; hence a partition 12 high will weigh 240 lb per lin ft of partition. As the joists are 16 in on c ters, each joist supports $1\frac{1}{3}$ lin ft of partition, weighing 320 lb. As this load concentrated at the middle span of the joists it is equivalent to a distribut load of 640 lb. In Example 4, we found the safe distributed load for the $2\frac{1}{2}$ 14-in joists of 22-ft span to be 2 970 lb. Subtracting 640 lb from this we h 2 330 lb, which may be used for the floor. As the floor-area supported by e joist is $29\frac{1}{3}$ sq ft, the safe strength of the floor per square foot is $2\,330/29\frac{1}{3} =$ lb, and the safe load is $79 - 15 = 64$ lb per sq ft. Hence the partition decrea the safe load by $86 - 64 = 22$ lb per sq ft. Whenever the upper-floor joi are supported by a partition carried by a floor below, the effect of the partiti and its load upon the strength of the lower floor should be very carefully co puted.

Bridging of Floor-Joists. By BRIDGING is meant a system of bracing floor-joists, either by means of small struts, as in Fig. 5, or by means of sin

[...]es of boards set at right-angles to the joists and fitting in between them. [...]e effect of this bracing is of decided advantage in sustaining any CONCEN-[...]ATED LOAD upon a floor; but it does not materially strengthen a floor to resist [...] UNIFORMLY DISTRIBUTED LOAD. The [...]dging also stiffens the joists, and pre-[...]nts them from turning sidewise. It is [...]tomary to insert rows of cross-bridging [...]m 5 to 8 ft apart; and to be effective [...]e rows of bridging should be in straight [...]es along the floor, so that each bridging-[...]ut may abut directly opposite those [...]acent to it. The method of bridging [...]wn in Fig. 5, and known as CROSS-[...]DGING, is considered to be by far the [...]st, as it allows the thrust to act [...]allel to the axis of the strut, and not [...]oss the grain, as must be the case [...]ere single pieces of boards are used. [...]e bridging should be of 1¼ by 3-in [...]ck, for 2 by 10-in and smaller joists, [...]d of 2 by 3-in stock for 12- and 14-in [...]sts.

Fig. 5. Floor-joists with Bridging

[...]raming of Wooden Floor-Beams. [...] dwellings, tenements and lodging-[...]ses it is frequently necessary to frame the timbers so that they are flush [...]h one another. The old methods of framing the tail-beams and headers or [...]ders and trimmers by mortise-and-tenon joints are now generally superseded [...] hanging the timbers in stirrups or malleable-iron joist-hangers. In this construction the entire strength of the timbers is retained, while the cost of the hangers is often less than the labor-cost in preparing the mortise-and-tenon joints. All headers 6 ft or more in length should be carried in joist-hangers or stirrups and this is usually required in the building codes of the large cities. In warehouses and all first-class buildings the framing should be done by means of joist-hangers. For light floors, with moderate spans, it is generally safe to frame the tail-beams into a header, provided the latter is strong enough to carry the load and allow 1 in in thickness for the mortising. Headers, [...], carrying not more than two tail-beams are often framed into the trimmers. [...] case the old methods of framing are used instead of the superior methods [...]h joist-hangers, the best shape and proportions for the tenons and ends of [...] tail-beams or headers are those shown in Fig. 6. This form of framing

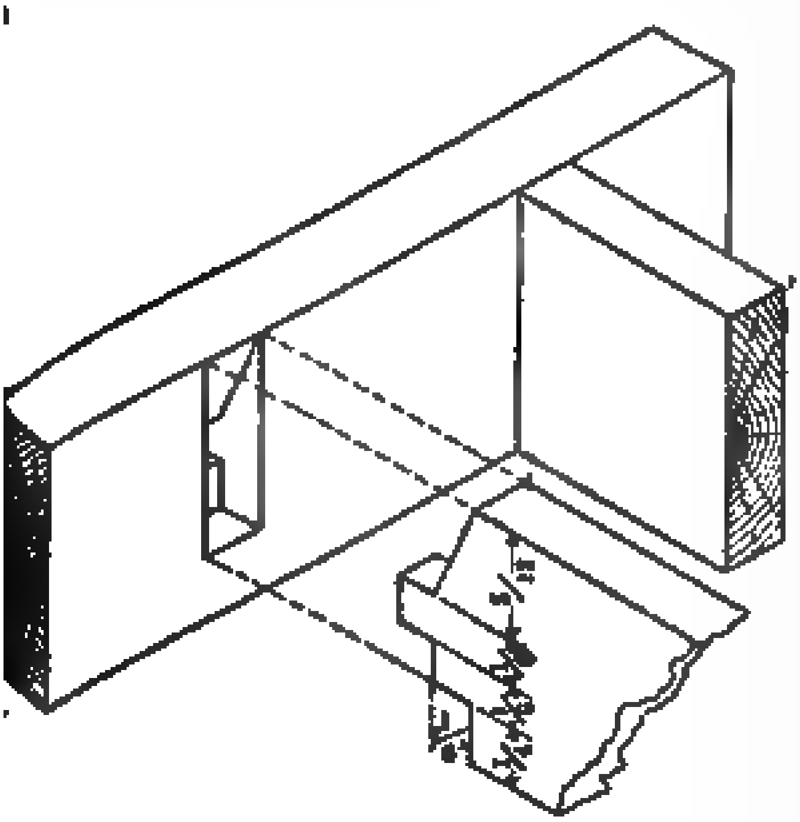


Fig. 6. Framing of Joists into Header

probably offers as large a proportion of the strength of the timbers as it possible to utilize, although for tail-beams it was the opinion of Mr. Kidd that a single tenon like that shown in Fig. 7 is fully as strong, especially wh the header is built up of 2-in planks spiked together. In either case, if the flo

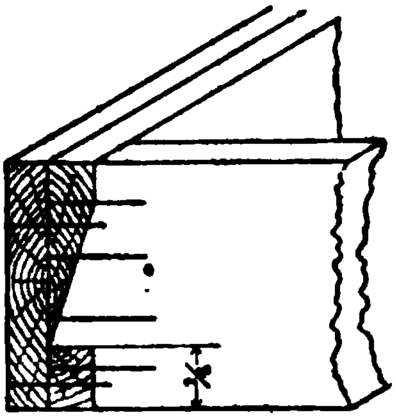


Fig. 7. Alternate Method of Framing Joists into Header

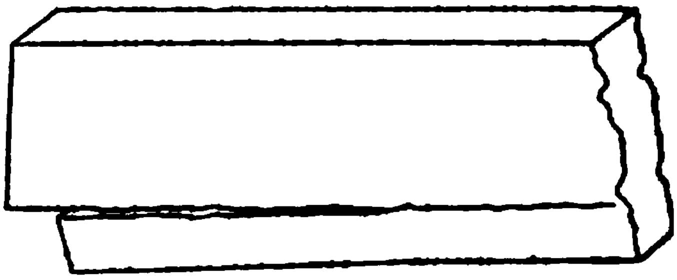


Fig. 8. Framed Joist Split by Load

is loaded to its full strength, the tail-beam will split at the bottom of t tenon, as shown in Fig. 8, which illustrates the weakening effect of t mortise-and-tenon framing.

Stirrups and Joist-Hangers. The first device used for framing headers trimmers without mortising was the wrought-iron stirrup shown in Fig. These are made either single or double, depending upon whether one or t beams are to be supported. To prevent the floor from spreading and thus p

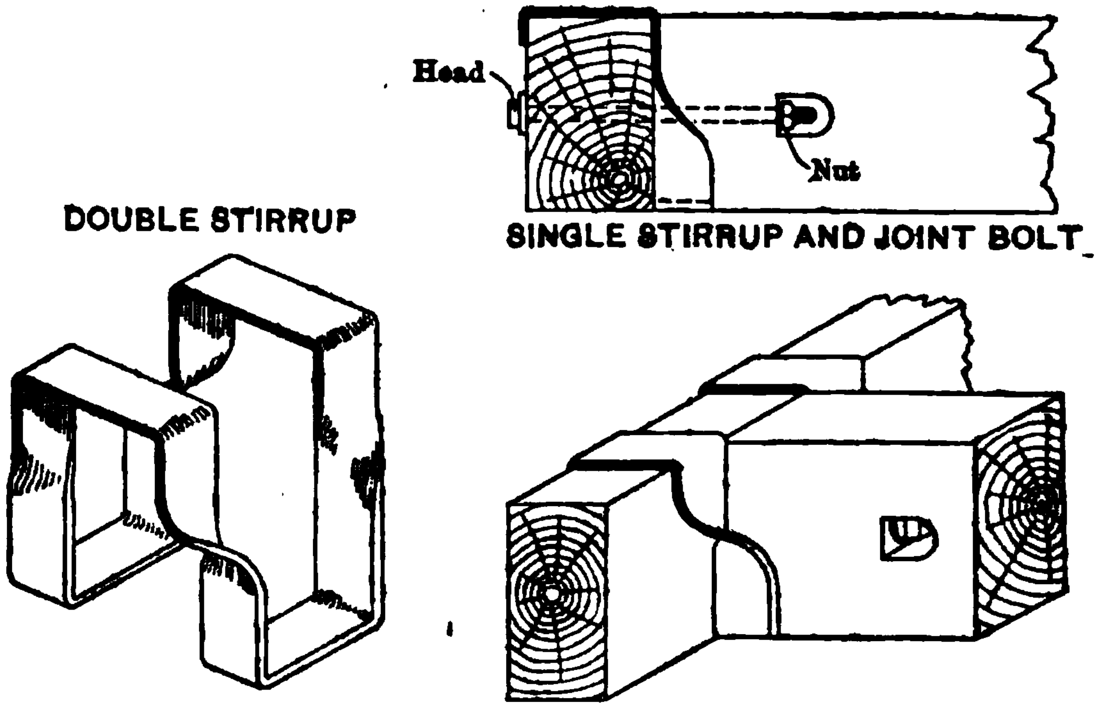


Fig. 9. Framing with Wrought-iron Stirrups

mitting the header to slip out of the stirrup, a joint-bolt may be inserted, shown in the two right-hand illustrations of Fig. 9. To determine the stren of a stirrup, multiply the sectional area of the iron, in square inches, by 12 lb per sq in. (Table 1, page 376.)

The following sizes of iron should, in general, be used for the different s of joists to be supported:

of joists or timbers to be supported, in inches	Sections of stirrup-iron in inches
2 by 8 to 3 by 10	¼ by 2¼
4 by 10 to 4 by 12	⅜ by 2½
6 by 12 to 3 by 14	⅜ by 3
8 by 12 to 4 by 14	½ by 3½
6 by 14	½ by 4
8 by 14 to 10 by 14	⅝ by 4

Joist-Hangers. Aside from the matter of strength there are objections to e use of stirrups. If the timber on which they rest is not perfectly dry, the irrups will settle by an amount equal to the shrinkage of the beam on which they t, and let down the header with them, and the projection of the iron above e top of the timbers will necessitate cutting out the flooring. If the stirrups e exposed in this way their appearance is objectionable. While they may designed to resist any tensional stress the resistance of steel to bending comparatively small, and the resulting crushing of the timber where they over the edge is the chief objection to the use of stirrups of this type heavily loaded floors. The small bearing of a timber on a stirrup is

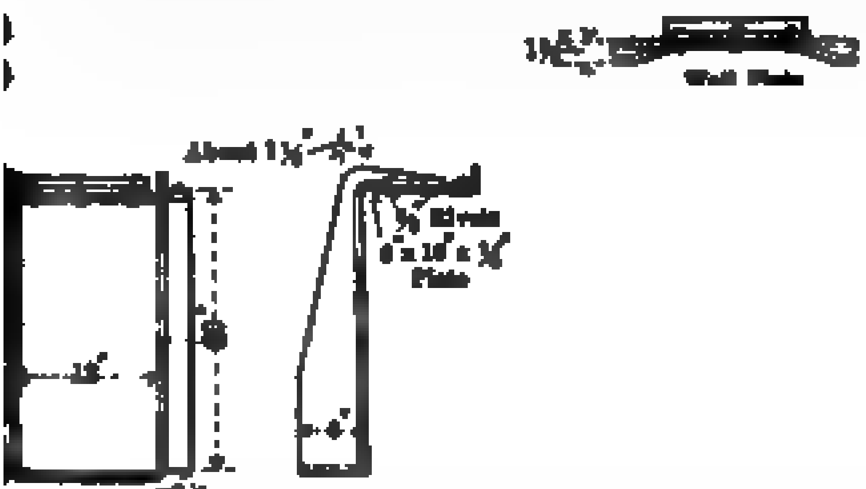


Fig. 10. Failure of Steel Stirrup Wall-hanger

sufficient to distribute the load on the wood over the required area. This eases the bearing per square inch, allows the hanger to crush into the edge tends to straighten out the stirrup as shown in Fig. 29, page 757. The e serious objection applies to the use of steel stirrup-hangers in brick walls arry beams free of the walls. As previously explained, all the load is ught to the extreme edge, causing a much greater load per square inch on masonry than is allowable. Fig. 10 * shows the effect of crushing, in a house-building in Minneapolis, Minn. Wall-hangers made of steel ps should not be used. Patented steel hangers riveted to bearing-plates likewise very undesirable as the crushing effect is greatest at the outer , due to the straightening-out tendency of the hanger at this point.

gs. 11 and 12 illustrate the Duplex and Goetz joist-hangers, which are nted and are claimed to be superior to the old-style stirrups. The Duplex er is used not only for ordinary building-construction, but for the most ily loaded mill-construction in factories and warehouses. As these hangers ade of malleable iron they will not straighten out when heated, in case of and drop the beams. That is what happens to wrought-steel stirrups

Taken from a paper on "Joist and Wall-Hangers," read by Mr. F. E. Kidder at a g of the Colorado Chapter of the American Institute of Architects, February 27,

when the twist becomes heated. This hanger has proven perfectly satisfact and is extensively used. Both are made in sizes to fit all regular sizes of joi or girders, and have ample strength for the purpose for which they are intend

Fig. 11. Duplex Joist-hanger

Fig. 12. Goetz Joist-hanger

As shown by the illustrations, they are made to be inserted in round holes bo in the side of the carrying timbers, at or a little above the center line. W these hangers the effect of shrinkage is reduced one-half, and the other two

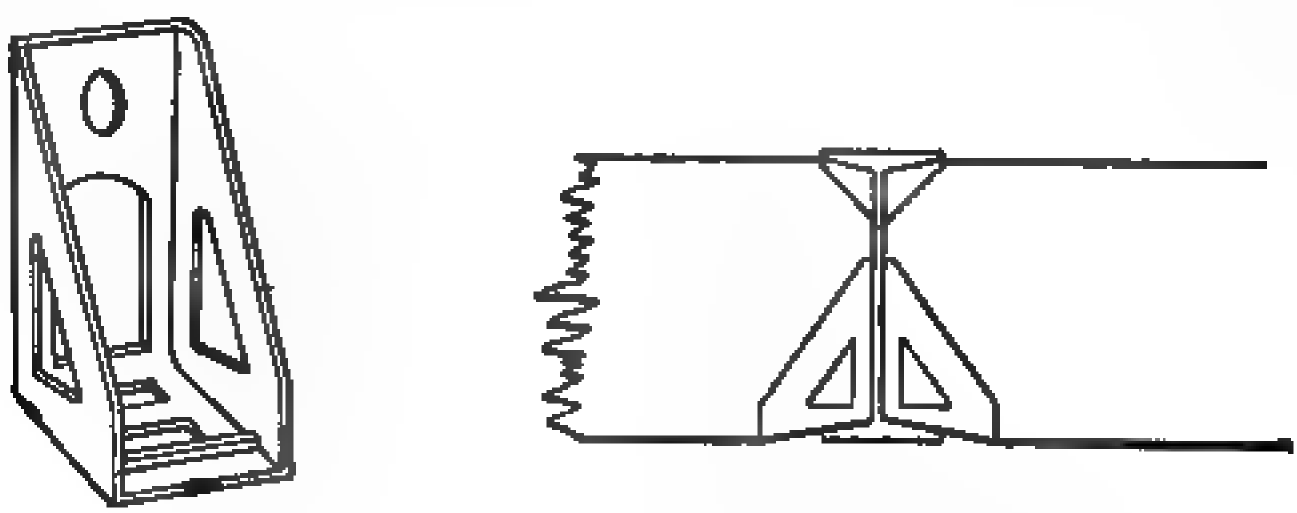


Fig. 13. Duplex I-beam Hangers

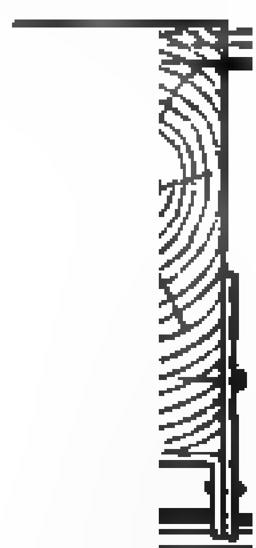


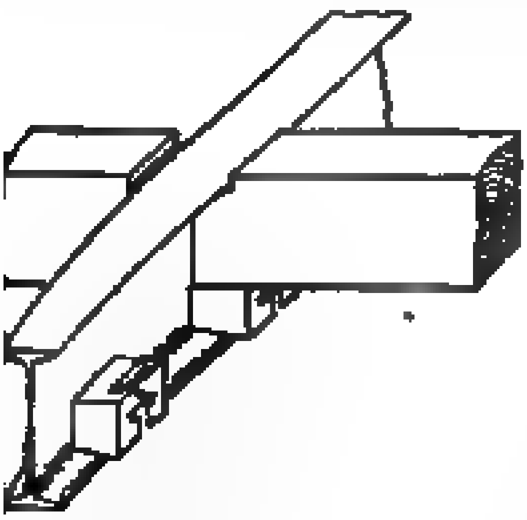
Fig. 14. Duplex I-beam Shelf-hanger. Joists Raised Less than Four Inches

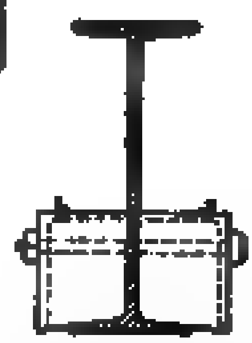
jections to the stirrup, previously mentioned, are overcome. The Du hanger has ridges on the inside of the side brackets to hold the beam.

For timbers of larger size and for the heaviest construction, the Duplex hang

wn in Fig. 32, page 789, are used and are bolted to the beams. By this struction the entire building is tied together laterally.

Fig. 13 shows the Duplex I-beam hanger for framing floor-joists to I beams. is hanger is made to exactly fit into the flange of the I beam. It has a rib





. 15. Duplex I-beam Box Hanger. Joists Raised More than Four Inches

Fig. 16. Duplex Wall-hanger for Joists

the bottom, ¾ in high, which serves as a tie when the joist is placed in the ger, and it provides a bearing of at least 4½ in for the joist. It is made carry any joist of regular size, and offers one of the best devices for framing oden joists to I beams of the same th. The hangers are bolted to the b of the I beam. Fig. 14 shows the plex I-beam shelf-hanger which is used en the construction requires the joists be raised above the lower flange of the am less than 4 in. Fig. 15 illustrates Duplex I-beam box-hanger and is mmended where the joists are raised re than 4 in above the lower flange the I beam. In both these construc- s the hangers are bolted singly or posite, as required, on the I beam and loads are carried on the lower flanges of the beams. Fig. 16 shows a ilar hanger made to support the wall-end of a floor-joist. This form of struction is considered much superior to the method of building the

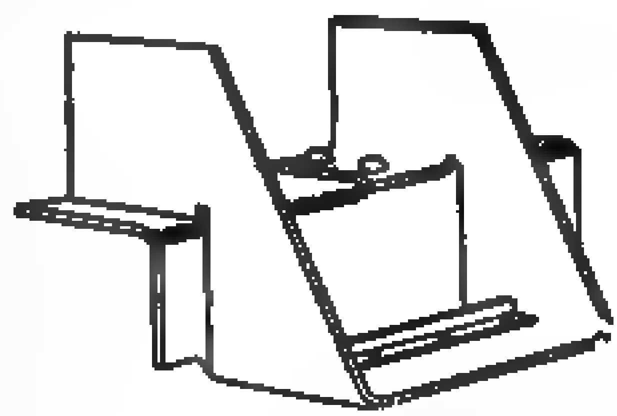


Fig. 17. Duplex Steel Wall-hanger for Large Beams

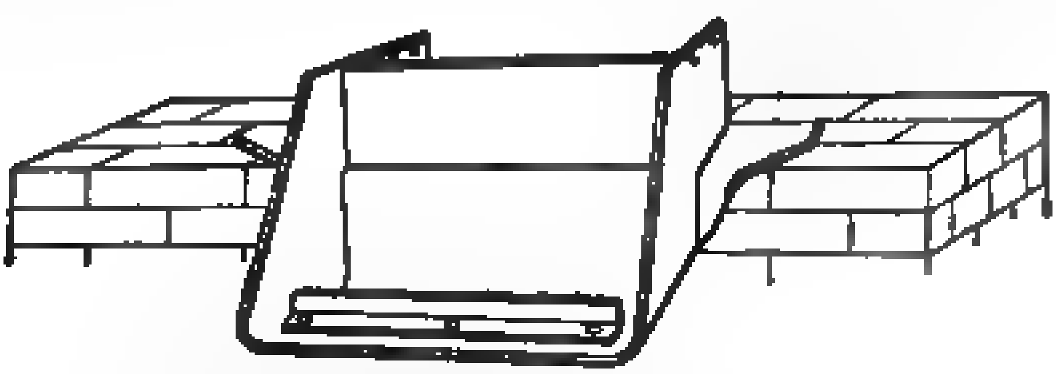


Fig. 18. Duplex Extra-heavy Wall-hanger for Mill-construction

ts into a wall, as it absolutely prevents dry-rot, and permits the joists to in case of fire, without throwing the wall. It also gives the load a good ing on the wall. Fig. 17 illustrates the Duplex steel wall-hanger for larger ms, and Fig. 18 shows the Duplex extra-heavy wall-hanger for the heaviest

mill-construction. These hangers bear the label of approval of the Natio Board of Fire Underwriters and are generally considered the best-designed w hangers now on the market. This hanger gives an extra bearing on the maso and is so constructed that it reacts as a unit and distributes the load equally o the entire surface of the masonry. There is no tendency for a hanger of t

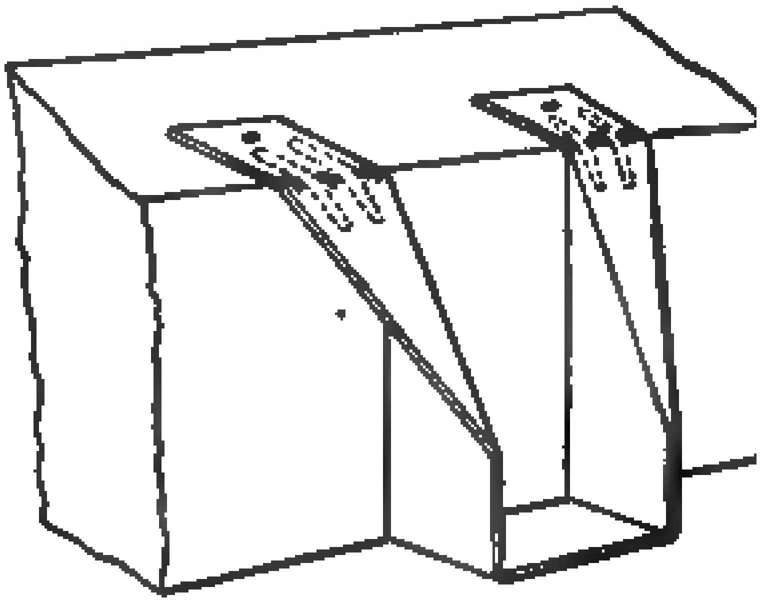
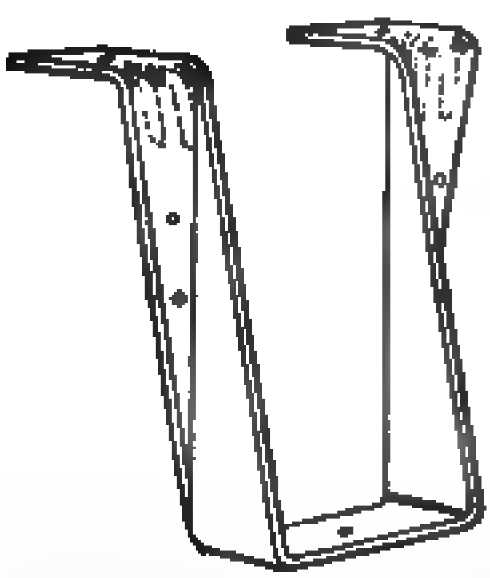


Fig. 19. Duplex Wall-hanger for Concrete Blocks

Fig. 20. "Ideal" Wrought-steel Beam-han

type to crush in at the edge of the masonry and straighten out, as is the c with some other types of wall-hangers. Fig. 19 shows the Duplex wall-han used in connection with walls constructed of concrete blocks. These hangers often used in repair-work in party walls, as they avoid the cutting of large h in the walls, and also provide an easy and simple method of carrying the jo clear of the walls. The Ideal hanger illustrated in Fig. 20 is made of wrou



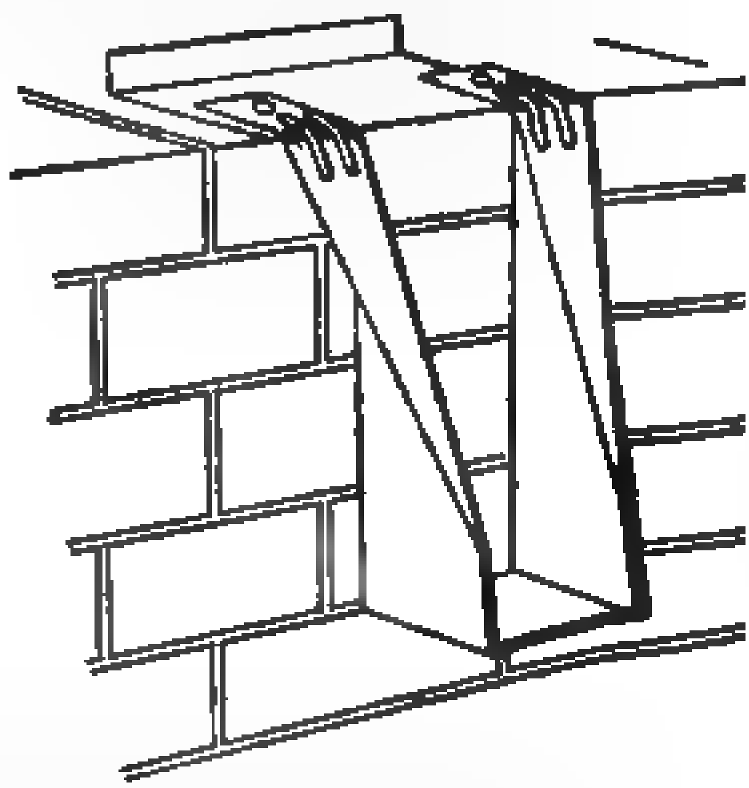


Fig. 21. "Ideal" Wrought-steel Beam-hanger

Fig. 22. "Ideal" Wrought-steel Wall-han

steel and corrugated at the points where it is bent over. This reinforces it tends to prevent bending at these points. Fig. 21 illustrates another for the Ideal hanger with holes for spiking to a timber. This hanger, als corrugated. In these hangers the full strength of the steel is retained as fibers of the metal are not cut in forming them. They are made of wrou steel bars folded to the required shape. Fig. 22 shows the Ideal hanger giv

a steel plate and in position to be built into a brick wall. Other illustrations wall-hangers are given in Chapter XXII. The Van Dorn hanger, illustrated Fig. 23, is essentially a stirrup forged from high-grade steel. The few tests it have been made would seem to indicate that it developes a greater re- ance to bending than the ordinary stirrup, while it gives a wider bearing

.23. Van Dorn Beam-hanger

Fig. 24. Van Dorn Wall-hanger

the joist and presents a much neater appearance. Fig. 24 shows the same er riveted to a bent iron plate, to build into brick walls. When the hanger o be used over a steel beam the upper ends are bent to fit over the flange of beam, as in Fig. 25. "Although I know of no test of the strength of a Van n I-beam hanger, it would seem as though it must be much stronger than

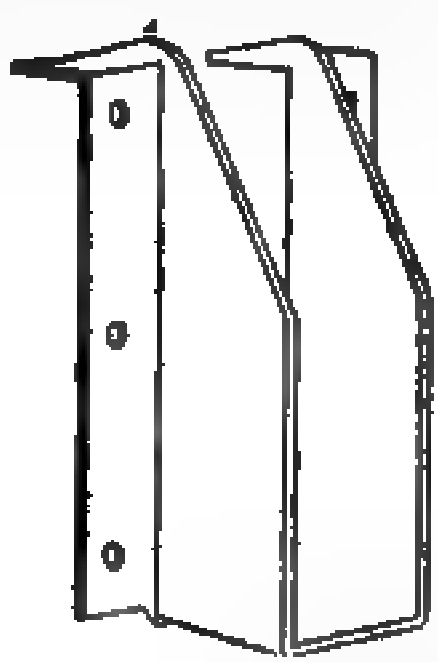


Fig. 25. Van Dorn I-beam Hanger

Fig. 26. National Joist or Beam-hanger

pattern made for wooden beams, on account of the clinch over the flange of I beam. The Van Dorn hangers have been used in many important build- s."*

. 26 and 27 show the general form of two other patented joist-hangers, h are forged from plate steel. Both of these hangers, also, are made to be

*F. E. Kidder.

built into brick walls and to go over steel beams. The National hanger (Fig.
has a flange on top, which helps materially in distributing the load over the
of the beam as shown in figure. The larger hangers of this style have hole
the top for large spikes. T
hanger and the Lane has
(Fig. 27) have been much u

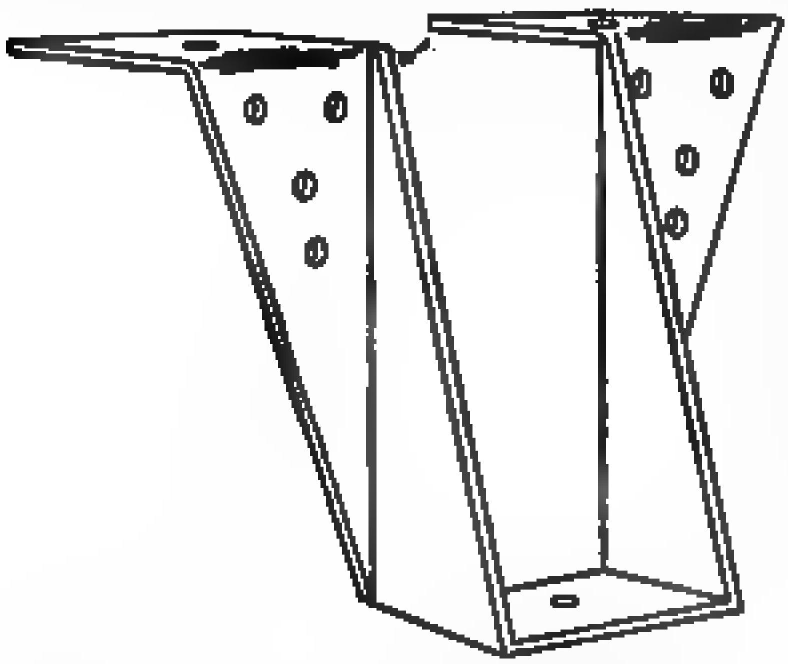


Fig. 27. Lane Joist or Beam-hanger

Comparative Strengths Different Types of Jo Hangers. Although the t
that have been made to de
mine the strength of diffe
hangers are few in number
sufficient number have b
made to show that any one
the hangers described, includ
the common stirrup, is ab
dantly strong for any SIM
FLOOR-BEAM not exceeding 4
14 in in cross-section. It
only in the case of a header
trimmer which supports a load over a considerable floor-area that the stren
need be considered at all. From tests
and on girder-hangers, it would appe
hangers usually develop great streng
by 14-in girder, sustained a load of
itself. A similar hanger held until
loaded up to 39 550 lb, when one
side broke off short under the
nipple projecting into the timber,
the condition of the hanger after
failure being shown in Fig. 28. A
common stirrup made from ⅝ by
2½-in wrought iron failed under a
load of 13 750 lb by bending and
pulling over the header, as shown
in Fig. 29. A 6 by 12-in steel
hanger "began to straighten out
under a load of 13 300 lb, and failed
to hold under a load of 18 750
lb."* SINGLE hangers of the
stirrup-type DO NOT BREAK, but fail
by the bending up of the parts
which lie over the top of the header

Fig. 28. Result of Test of a Two-p
Beam-hanger

as shown in Fig. 29. They also appear to crush the wood under them
ticularly at the edges, to a very much greater extent than does the spoo
the Duplex hanger. With a DOUBLE stirrup the ultimate strength is meas
by the strength of the iron. Thus, a double stirrup, made of ⅝ by 2¼-in wro
iron, was loaded up to 57 650 lb (28 825 lb on each side), when it broke at
of the lower corners. A single stirrup would of course be just as strong
could be kept from bending. In actual construction the flooring over the be
to some extent prevents the top of a stirrup from springing up. The tests

* From data compiled by Mr. Kidder from a series of tests on beam-han
and joist-hangers.

been made of two-part hangers show conclusively that where only a e hanger is used the holes h are bored in the header do seriously affect its strength the load is within the safe , and a test made at Balti- , Md., August 24, 1904, with 12-in joists, spaced 12 in on rs and suspended by these rs let into a header formed ree 3 by 12-in joists, spiked her, would seem to prove even when the holes are 12 in they do not seriously weaken The only record of the fail- f any form of hanger when tual use in a building, of I am aware, is that of a e in Minneapolis, where a n of six floors of a ware- fall, on Nov. 7, 1902, through ilure of a wall-hanger made a 4 by 2 by ¼-in structural- angle, which was sheared and bent, and riveted to an 8 by 16 by ¼-in g-plate. The failure was due to the crushing of the outer edge of the ork under the hanger, and the consequent bending up of the top portion. ctual load on the hanger was about 15 000 lb." *

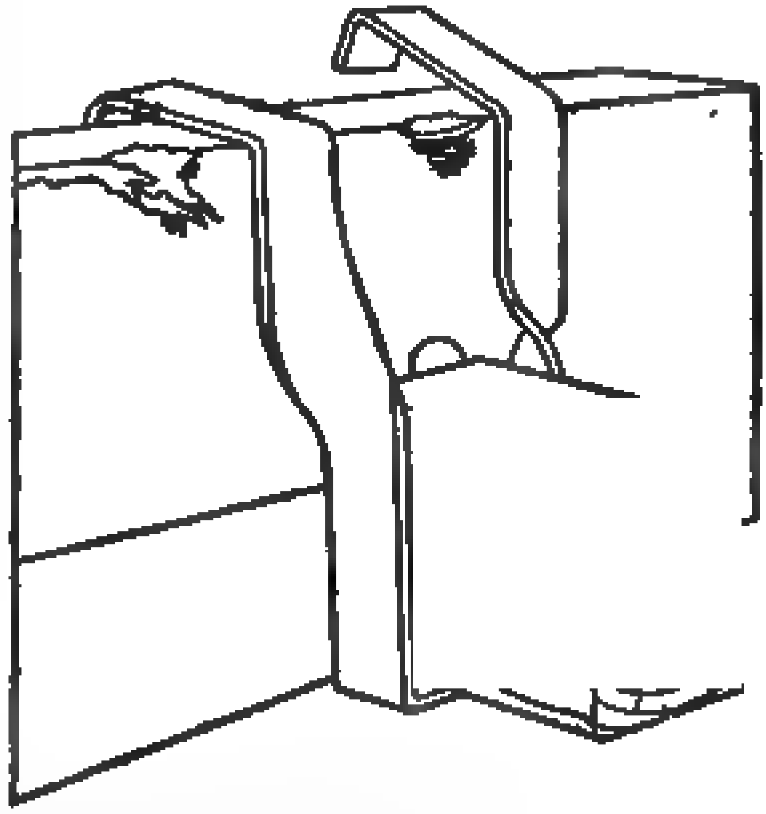


Fig. 30. Result of Test of Wrought-iron Stirrup-hanger

* F. E. Kidder. See, also, Engineering News, Nov. 20, 1902.

CHAPTER XXII

WOODEN MILL AND WAREHOUSE-CONSTRUCTIO

By

A. P. STRADLING

SUPERINTENDENT OF SURVEYS, PHILADELPHIA FIRE UNDERWRITERS' ASSOCIATION

1. Mill-Construction

Definition. The term MILL-CONSTRUCTION is commonly used to design method of construction brought about largely through the influence of Boston Manufacturers' Mutual Fire Insurance Company of Boston, Mass., especially through the efforts of Mr. Wm. B. Whiting, whose judgment mechanical matters, and experience and skill as a manufacturer were for m years devoted to the interests of insurance companies, and to the improvem of factories of all kinds. The extended use of this system and the improvem that have been made in it during recent years are probably due more to influence of Mr. Edward Atkinson, President of the Boston Manufactu Mutual Insurance Company and Director of the Insurance Engineering Exp ment Station at Boston, than to that of any other individual.

Cost. The purpose of mill-construction is to reduce the fire-risk to its l est point without going to the expense of fire-proof construction. The increa cost of heavy timber, however, and in fact of all lumber, together with lessened cost of the erection of the so-called FIRE-PROOF TYPES, constru entirely of reinforced concrete, or built with protected steel frames and inc bustible floors, and the recognition, also, of the obvious advantages of n FIRE-RESISTING CONSTRUCTION, especially in the congested sections of cities, bringing these types into more general use. The cost of these latter type construction is, in many instances, no more than the cost of various type mill-construction.

The Slow-burning or Mill-Construction Type. The experience of y has entirely justified the use of this type. It renders possible a somewhat costly, and at the same time, what is of great importance, a more effective sys of fire-protection than can be installed in buildings of light construction, the so-called JOISTED FLOORS and with the roofs made of boards supporte 2-in, 3-in, or 4-in joists. The entire subject of SLOW-BURNING or MILL-C STRUCTION as applied to factories is most admirably described and illustr in Report No. 5 of the Insurance Engineering Station of the Boston Ma facturers' Insurance Company, No. 31 Milk Street, Boston, Mass., from w the author has, by permission, taken and adapted many of the following i trations and descriptions.

2. What Mill-Construction Is*

(1) **Heavy Timbers.** MILL-CONSTRUCTION consists in so disposing timbers and planks in heavy, solid masses as to expose the least numb corners or ignitable projections to fire; and to the end, also, that when occurs it may be most readily reached by water from sprinklers or hose.

* From Report No. 5 of the Insurance Engineering Station of the Boston Man turers' Insurance Company, No. 31 Milk Street, Boston, Mass.

(2) **Fire-Stops.** It consists in separating every floor from every other bor by incombustible STOPS, by installing automatically closing hatchways nd by encasing stairways either in brick or other incombustible partitions, so hat a fire will be retarded in passing from floor to floor to the utmost consistent ith the use of wood or any material not absolutely fire-proof.

(3) **Fire-Retardants.** It consists in guarding the ceilings over all specially azardous stock or processes with FIRE-RETARDANT MATERIALS, such as plastering laid over wire lath or expanded metal, or over wooden dovetailed lath, llowing the lines of the ceilings and of the timbers and leaving no interspaces etween the plastering and the wood; or else in protecting the ceilings over azardous places with asbestos, air-cell boards, sheet metal, Sackett Plaster oard, or other fire-retardant.

(4) **Fire-Safeguards.** It consists not only in so constructing the mill, workhop, or warehouse that fire will pass as slowly as possible from one part of the ilding to another, but also in providing all suitable SAFEGUARDS AGAINST FIRE.

2. What Mill-Construction Is Not

(1) **Concealed Spaces.** Mill-construction does not consist in so disposing a en quantity of materials that the whole interior of a building becomes a SERIES WOODEN CELLS, or concealed spaces, connected with each other directly or cracks through which fire may freely pass where it cannot be reached by ater.

(2) **Size of Timbers, Fire-Stops, etc.** It does not consist of an open-timber struction of floors and roofs which resembles mill-construction, but which built with light timber of insufficient size and with thin planks, without fireps or fire-guards from floor to floor.

(3) **Stairways.** It does not consist in connecting floor with floor by COMSTIBLE WOODEN STAIRWAYS encased in wood less than two inches thick.

(4) **Partitions.** It does not consist in putting in very numerous LIGHT, OODEN DIVISIONS or partitions.

(5) **Sheathing and Furring.** It does not consist in SHEATHING brick walls ith wood, especially when the wood is set off from the walls by FURRING, and en if there are stops behind the furring.

(6) **Varnish.** It does not consist in permitting the use of VARNISH on woodrk over which a fire will pass rapidly.

(7) **Glass, Fire-Shutters and Wire-Glass.** It does not consist in leaving ndows exposed to adjacent buildings and unguarded by FIRE-SHUTTERS or RE-GLASS.

(8) **Painting and Dry-Rot.** It does not consist in painting, varnishing, ng or encasing heavy timbers and thick planks, as they are customarily livered, and thus making possible what is called DRY-ROT, caused by a lack ventilation or opportunity to season.

(9) **Sprinklers, Pumps, Pipes, Hydrants, etc.** It does not consist in ring even the best-constructed building in which dangerous occupations are lowed without AUTOMATIC SPRINKLERS, and without a complete and adequate ipment of PUMPS, PIPES and HYDRANTS.

(10) **Finishing in Wood and Other Materials.** It does not consist in ng more WOOD IN FINISHING a building after the floors and roof are laid than bsolutely necessary, since there are now many safe methods available at low t for finishing walls and constructing partitions with slow-burning or in-

combustible materials. Accordingly if plaster is to be put on a ceiling and to follow the line of the underside of the flooring and the flooring-timbers, should be PLAIN LIME-MORTAR PLASTER, which is sufficiently porous to pen seasoning. The addition of a skim-coat of lime-putty is hazardous, especi if the overflooring is laid over rosin-sized or asphalt paper. This rule app to almost all timber as now delivered. Examples of all of the faulty meth of construction above mentioned have been found in various buildings p porting to be of mill-construction, and they all form parts of what has so times been called COMBUSTIBLE CONSTRUCTION.

4. Standard Mill-Construction

Example of Standard Mill-Construction. Fig. 1 shows a cross-sect through a mill of the customary or STANDARD TYPE recommended by the Bos Manufacturers' Mutual Insurance Company, the details of construction be revised to May, 1908.

Walls. If additional stories are required, the walls may be increased thickness according to the number of stories added, after a computation been made of the loads which a STANDARD FACTORY may be called upon to s tain. Walls should be of brick and at least 13 in thick in the upper story, their thickness should be increased in the lower stories to support additio loads. Plastered walls are often to be preferred to unplastered walls. Wind arches and door-arches should be of brick, and window-sills, outside door-sills under-pinning of granite or concrete.

Roofs and Floors. The roofs should be of 3-in pine planks spiked dire to the heavy roof-timbers, and covered with five-ply tar-and-gravel roofi Roofs should incline from ½ to ¾ in per ft, and incombustible cornices recommended when there is exposure from neighboring buildings. Fl should be of spruce planks, 4 in or more in thickness according to the fl loads, spiked directly to the floor-timbers, and kept at least ½ in away from face of the brick walls. In order to obviate the danger of cracking the wa which sometimes results from the swelling of planks laid close against th these spaces left between walls and floor-planks must be covered by strip battens both above and below. In floors and roofs, the bays should be fr 8 to 10½ ft wide, and all planks two bays in length should be laid to br joints every 4 ft, and grooved for hard-wood splines. Usually an overfloo birch or maple is laid at right-angles to the planking, but the best mills hav double overfloor, a lower one of soft wood, laid diagonally upon the planks an upper one laid lengthwise. This latter method allows boards in alleys passageways to be easily replaced when worn, while the diagonal boards br the floors, reduce the vibration, and distribute the floor-loads more unifor than the former method. Between the planking and the overfloor should two or three layers of heavy, hard paper, laid to break joints, and each mop with hot tar or similar material to make a reasonably water-tight as wel dust-tight floor. The usually rapid decay of the basement or lower floor mills makes it desirable, whenever wood is not absolutely necessary, to m such floors of cement. If wooden floors are required, crushed stone, cind or furnace slag should be spread evenly over the surface, and covered wit thick layer of hot-tar concrete. On this tarred felt is often laid, well mop with hot-tar asphalt, and over it a flooring of 2-in seasoned planks, well pres down and nailed on edge without perforating the water-proofing under it. hard-wood boards of the overfloor are then nailed across the planks. Cem concretes promote decay of wood in contact with them. If extra supports required for heavy machinery, independent foundations of masonry should

Fig. 1. Modern Mill-building of the Standard Type

provided. In view of the difficulties frequently met with in preserving ba ment floors of the ordinary timber construction, because of the lack of suital ventilation underneath, and also in view of the rapid decay of timber and pla floors in bleacheries, dye-works, print-works, and the like, in which the flo quickly become saturated with moisture, artificial-stone floors are being l in many of the modern plants.

Sizes and Kinds of Timbers. All woodwork, not STANDARD CONSTRU TION, in order to be SLOW-BURNING, must be in LARGE MASSES which present t least surface possible to a fire. No pieces less than 6 in in width should be u for the lightest roofs, and for substantial roofs and floors much wider ones a needed Timbers should be of sound, long-leaf, yellow pine, and for sizes to 14 by 16 in, single pieces are preferred, or, timbers 7 to 8 by 16 in, are oft used in pairs bolted together, without air-spaces between. They should n be painted, varnished or filled for three years because of the danger of dry r and for the same reason, an air-space should be left in the masonry around t ends.

Beam-Boxes, Column-Caps, etc. Timbers should rest on CAST-IR PLATES or BEAM-BOXES in the walls and on cast-iron caps on the columns. BEA BOXES are of value as they strengthen the walls when the floor loads are hea and the distance between windows small; they facilitate the laying of t bricks and the handling of the beams; and there is less danger of breaking t bricks in putting the beams in place. They also insure proper air-spaces arou

Fig. 2. Floor-timber on Wall-plate

Fig. 3. Roof-timber on Wall-plate

the ends of the beams. Fig 2 shows a floor-timber resting on a CAST-IRON WA PLATE with a lug for anchoring the timber to the wall. Fig. 3 shows a ro timber resting on a CAST-IRON WALL-PLATE, an overhanging, open, wood cornice and a wrought-iron joist-anchor. Fig. 4 shows a CAST-IRON CAP a PINTLE for columns, and dogs for holding the floor-timbers together. Fig. shows a roof-timber resting on a COLUMN-CAP cast to fit the slope of the ro the timbers are held together by 1-in wrought-iron dogs. These diagrams a intended only as general illustrations of SLOW-BURNING or MILL-CONSTRUCTI The details should always be adapted to the special conditions of the site a to the purposes for which the buildings are used.

Columns of yellow pine should be bored through the axis, making a 1½- diameter hole, and should have ½-in lateral vent-holes near the top and botto The ends should be carefully squared. To prevent dry-rot, WOODEN COLUM

uld not be painted until they are thoroughly seasoned. They should be set pintles which may be cast in one piece with the cap, or separately. Cast-iron columns are preferred by some engineers, and when a building is equipped th automatic sprinklers, such columns have proved satisfactory; but they

Fig. 4. Post-cap and Pintle for Floor-timber and Columns

Fig. 5. Roof-timbers on Column-cap

not as fire-resisting as wooden columns. Wrought-iron or steel columns uld not be used unless encased with at least 3 in of fireproofing.

Windows should be placed as high and made as wide as possible to obtain the atest amount of light, and the use of ribbed glass is recommended for the per sashes.

Weight, Deflection and Vibration. In computing the size of the timbers ratio to the working-load, consideration must be given not only to the weights ich are to be carried, but also to the character of the machinery which is be operated on the floors. Beams of sufficient strength to support the weights y vibrate or deflect under the weight and action of the machinery; and there , therefore, three factors, weight, deflection and vibration, which must considered in determining the width and depth of the beams that are to used in the structure.

Objectionable Types of Construction. "We do not approve what has n sometimes miscalled mill-construction, that is, longitudinal girders ting upon posts and supporting floor-beams spaced 4 ft, more or less, on ters. This mode of construction not only adds to the quantity of wood d, but the disposal of the timbers obstructs the action of the sprinklers, prents the sweeping of a hose-stream from one side of the mill to the other, and girders also obstruct the most important light, that from the top of the ndows."

Timber, Ventilation, Painting, etc. Timbers, unless known to be thorghly seasoned, should not be encased in any kind of air-proof plastering nor nted with oil-paints; white-wash, calcimine and water-paints may be used, they are porous. As a rule, timbers should be left unprotected, since a which will seriously impair and destroy heavy timbers will already have its work upon other parts of the structure.

Single and Compound Beams. While, in general, single beams should be d, in some instances it may be desirable to substitute compound beams, de by fastening two or more beams or thick planks side by side. It is often er to obtain well-seasoned lumber in small dimensions. Such compound ms should be tightly bolted together without air-spaces, and owing to the ger of dry-rot, should not be painted or varnished for three years.

Steam-Pipes. If a mill is to be heated by conveying steam through pi such pipes should be hung overhead.

Cornices. Wherever buildings are exposed or are liable to be exposed fire in the near future, the cornices should be of non-combustible construction preferably, the walls should extend above the roof-timbers.

Glass, Frames and Shutters. All openings in walls should be protec either by approved wire-glass in approved, metal frames or by standard f shutters.

5. Belts, Stairways and Elevator-Towers

Continuous Floors. One of the most important features of SLOW-BURN CONSTRUCTION is to make each and every floor CONTINUOUS from wall to w

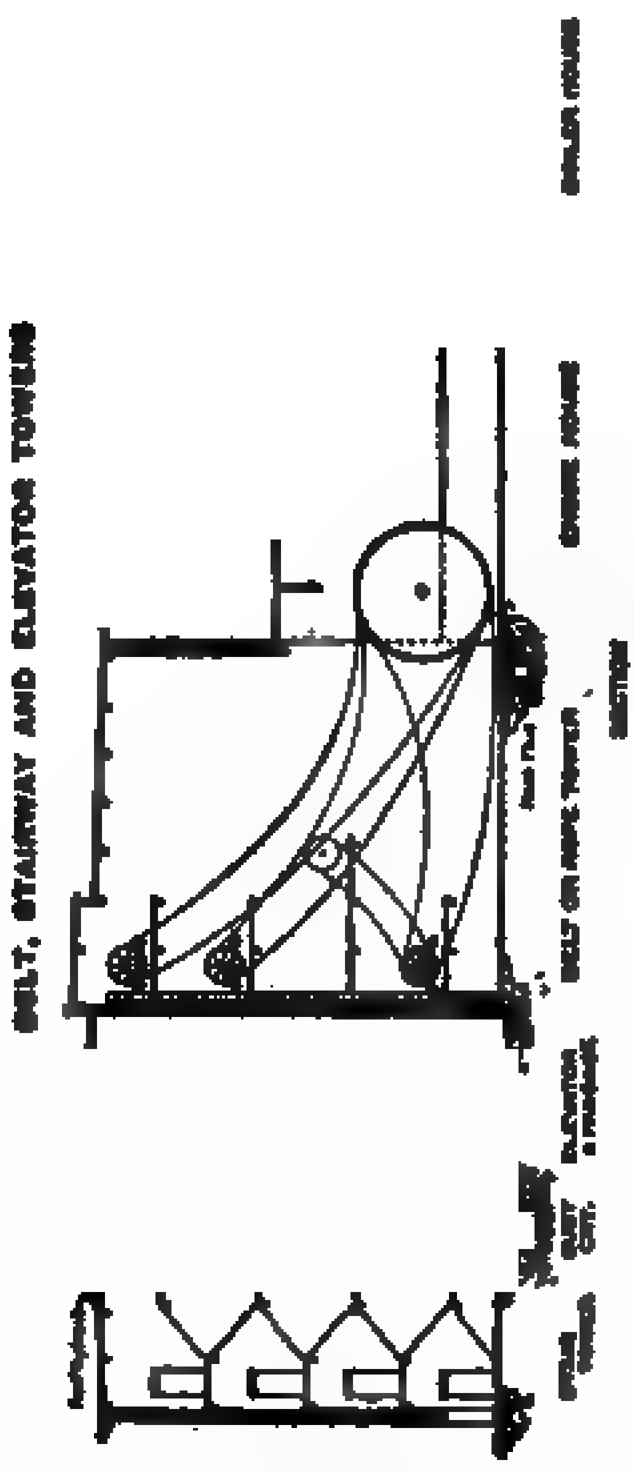


Fig. 4. Section through Tower for Elevators, Stairs and Belts

avoiding, as far as possible, holes for bel fire may be confined to the story in whi owner, engineer or builder will, therefore main belts, in BRICK TOWERS or in sections

y incombustible walls. All openings in these walls should be protected by STANDARD FIRE-DOORS, preferably self-closing. In modern practice all belts and ropes which may be used for the transmission of power to the various rooms, be placed in INCOMBUSTIBLE VERTICAL BELT-CHAMBERS, from which the power transmitted by shafts through the walls into the several rooms of the factory. here should be no unprotected openings in the inner walls of this BELT-HAMBER.

Shafts above Roof. Skylights. All SHAFTS for STAIRS, ELEVATORS, ELTS, etc., should extend at least 36 in above the roof, and all such shafts ould be, if possible, on the outside of the building. Elevator and belt-shafts ould be covered with thin glass skylights in metal frames, protected underneath with wire netting. Figs. 6 and 7 illustrate a section and plan of a COTTON-LL, showing elevator, stair and belt-shafts arranged on the above principle. LOSETS should be in a separate tower rather than in manufacturing rooms.

The Boiler-Plant should be in a separate building cut off from the engine-om by a brick wall, and the openings in this wall should be protected by AUTO-TIC, SLIDING, STANDARD FIRE-DOORS.

6. Standard Storehouse-Construction

xample of Storehouse-Construction. Fig. 8 shows a cross-section through fire-tower and Fig. 9 the first-story plan, including the elevator and stair-er of a four-story ehouse.

rea. Buildings this purpose ld not, in gen-, exceed 5 000 sq in AREA. When d, however, for ge of non-haz-us goods, the may be in-sed to 10 000 sq

Fig. 8. Four-story Storehouse. Section through Fire-tower

ight of Stories. storehouses, the s should be made low enough (Fig. 10) to prevent overloading, and when ed for case-goods, the HEIGHT OF STORIES should be sufficient to take two , with a 12-in. clear space under the beams to allow for the distribution of r from the sprinklers.

-Walls. For convenience, as well as to separate the different hazards of aterials and finished goods, the building should be divided into sections E-WALLS extending at least 36 in above the roof.

-Story Storehouses. A ONE-STORY STOREHOUSE is recommended in ce to the design just described, whenever there is a sufficient quantity l land at disposal for this purpose. The one-story building is cheaper, convenient, and, when separated into small divisions by fire-walls, repre-he safest method of storehouse-construction.

bers and Framing. The FLOOR-TIMBERS and ROOF-TIMBERS should be -leaf yellow pine, in single pieces, if possible. If necessary to use double , they should be bolted together without air-spaces between them. Tim-

bers should rest on cast-iron plates or beam-boxes in the walls, and on cast-ir caps on the columns. At least ½-in air-spaces should be left around all bea built into the masonry, allowing free ventilation and preventing dry rot. Co

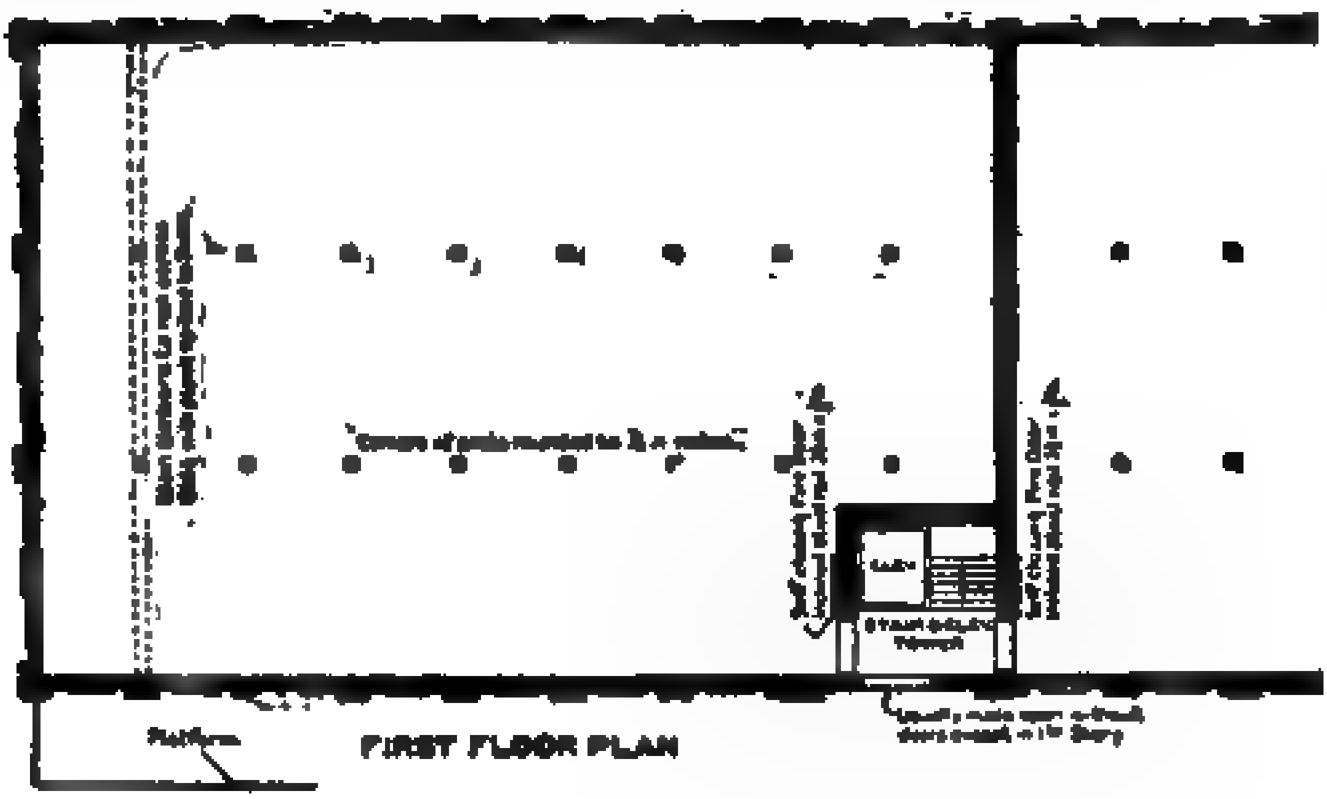


Fig. 9. Four-story Storehouse. First-story Plan

umns of yellow pine should have their end-surfaces cut square with the colum axis.

Floors. The FLOORS of such buildings should be continuous, without op ings, and of the standard slow-burning construction, described under STAND

Fig. 10. Four-story Storehouse. Isometric View

MILL-CONSTRUCTION. The flooring should be constructed as called for u STANDARD MILL-CONSTRUCTION. In order that the floors may be as ne water-proof as possible, tarred paper, mopped with tar, should be applie previously suggested. The floors in each story of the tower should be at l 1 in lower than the floor in the adjoining compartment, and the sills of the d openings to the tower should be inclined to make up the difference in le The sill, also, of the outside door of the tower should be lower than the to floor.

Scuppers. Water on the floors of the tower will ordinarily flow down the tower-stairs, and the arrangement of the floor-levels indicated above will ordinarily prevent water from an upper story from flowing into one of the lower compartments, if it is escaping through the tower. Cast-iron SCUPPERS are advised, and they should be set in the brickwork at frequent intervals, and so designed that they will carry away rapidly a maximum quantity of water from the floors of each compartment. To further the drainage of water, the floors should be inclined from the middle of the compartments to the scuppers. Fig. 11 shows the WIND-SHIELD SCUPPER* which embodies the latest improvements.

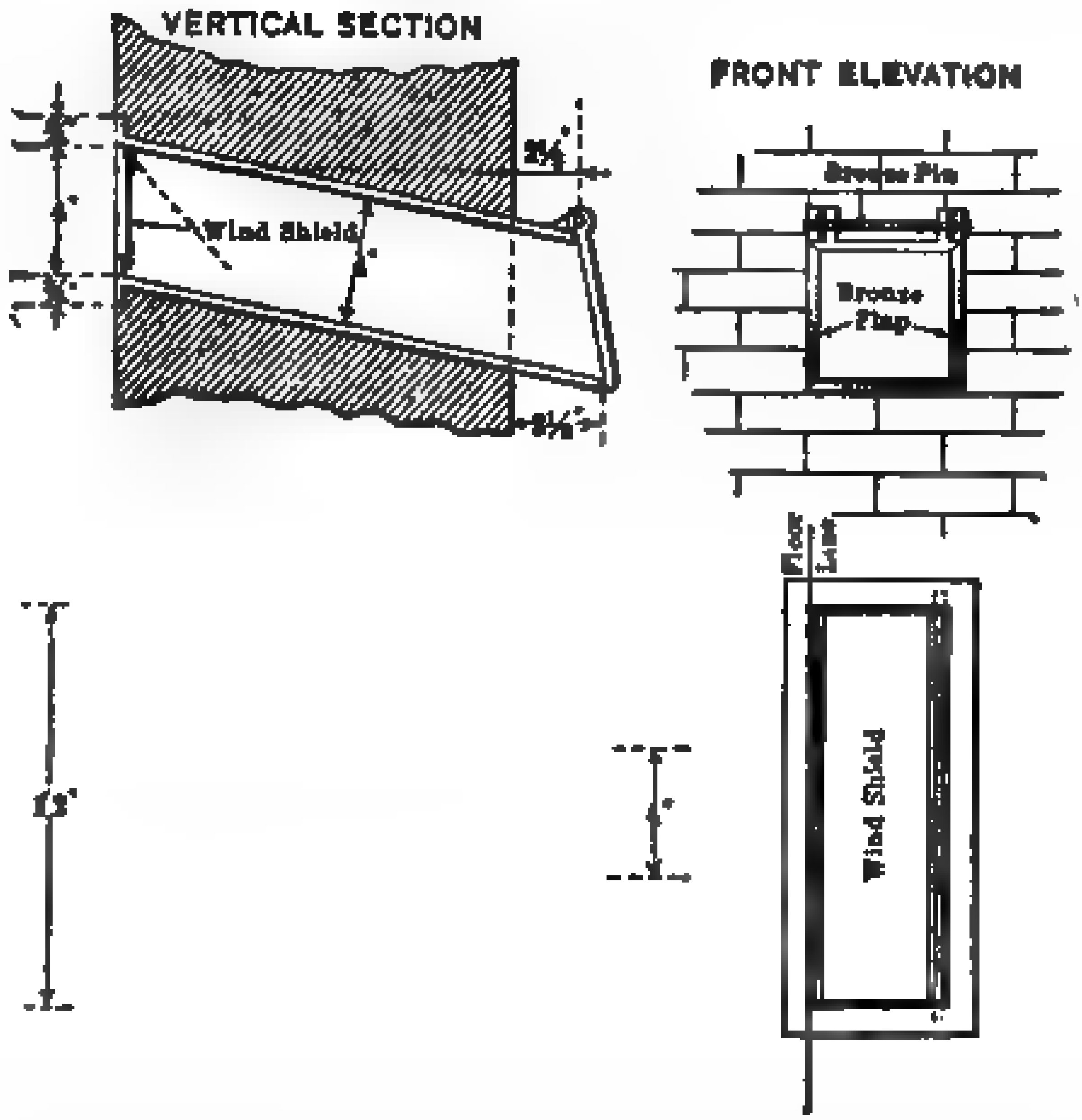


Fig. 11. Detail of Wind-shield Scupper

In the old-style scupper only one flap is provided on the outside of the building. During winter and windy weather, this flap blows open and sometimes freezes open. This results in a continuous draft through the scupper and over the working floor of the factory or warehouse and necessitates an increase in the amount of heat furnished. The scupper shown in Fig. 11 corrects this condition by providing the light wind-shield on the floor-level of the scupper. When the outer flap blows open the wind-shield shuts off the draft from the outside. The scupper, in addition, acts as a fire-retardant when an adjoining building is burning, and when there is a tendency for the flames to communicate through the open scupper and ignite merchandise on the floor. The wind-shield, by shutting off the drafts and fire, acts as a retardant or shield to keep out the flames.

* Manufactured by the Wind-Shield Scupper Company, 1 Madison Avenue, New York City.

Tower for Stairways, Elevators, etc. Access to the various stories obtained by means of a BRICK TOWER outside the main building, extending 36 above the roof, and containing STAIRWAYS, ELEVATORS, ETC., access to which

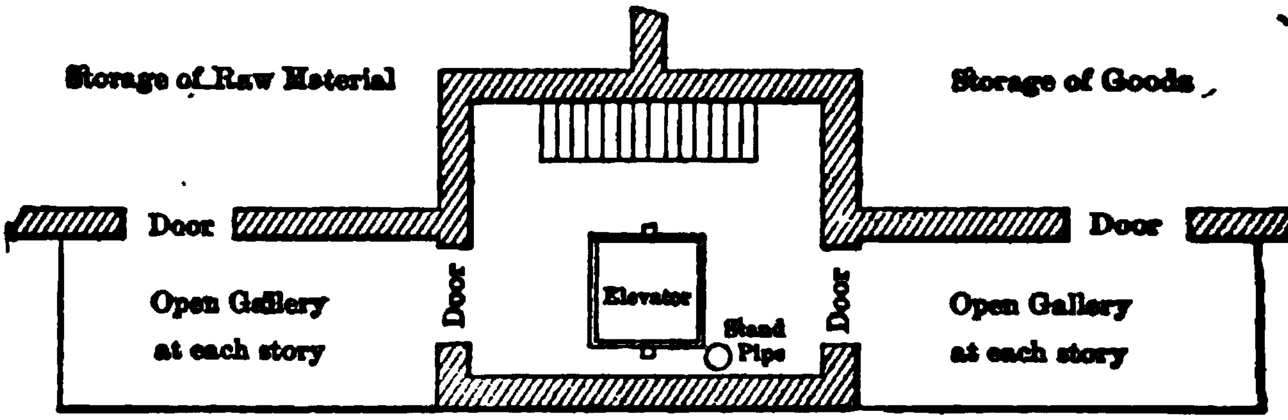


Fig. 12. Stairway-tower and Galleries at Side of Storehouse

obtained by open galleries at each floor-level. (See Fig. 12.) A door from the upper story of the tower affords a ready means of reaching the r AUTOMATIC HATCHES are not necessary for the elevator, as GUARD-GATES s every purpose. If it is necessary to construct the tower for the elevator stairs inside of the building, access to it should be as shown in Fig. 13. T

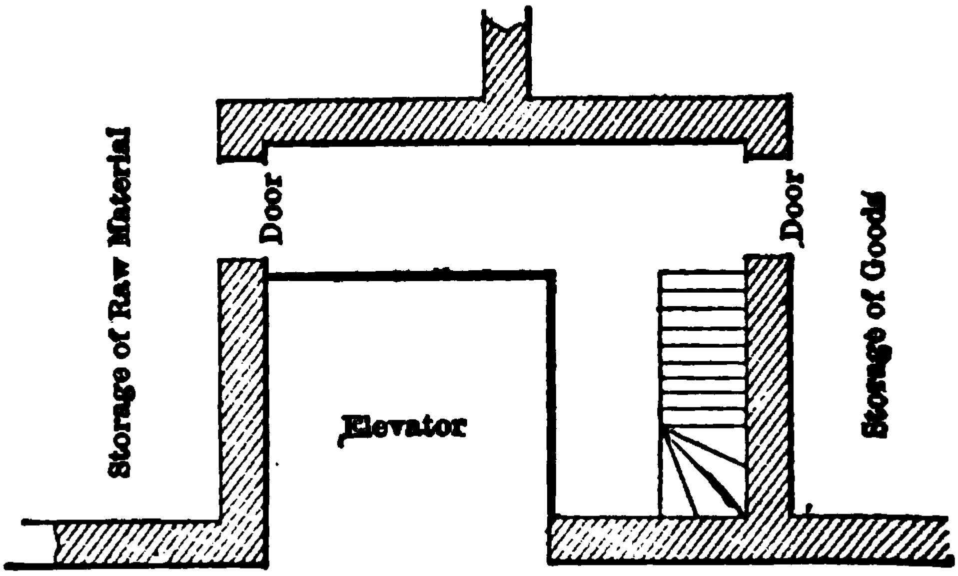


Fig. 13. Stairway-tower Inside of Storehouse

construction serves, also, as a FIRE-TOWER, part of the outside wall omitted.

Roof Walls and Parapets. The WALLS should extend 36 in above the and the PARAPET should be laid in cement, because the moisture readily sorbed by the bricks would otherwise pass downward and make the walls top story damp. In some instances a course of bricks dipped in coal-t laid above the roof-level.

Sprinklers, Standpipes and Hose. Mills and storehouses should be tected throughout by AUTOMATIC SPRINKLERS and by inside STANDPIPE HOSE-EQUIPMENTS. Dry-pipe sprinklers should never be used unless it practicable to heat the building. These systems should be planned and vised by a thoroughly reliable fire-protection engineer. (See, also, Ch XXIII, pages 903 to 905.)

7. Example of One-Story Work-Shop

conomy. For work-shops on cheap, level land, and especially for buildings which the stock is heavy, ONE-STORY BUILDINGS have proved to be more nomical than higher buildings, in cost of floor-area, supervision, moving ck in process of manufacture and repairs to machinery, much of which be run at greater speeds than when it is in high buildings.

Warming and Ventilating. Window-Area. Such buildings are readily med and ventilated, and heavy-plank roofs are free from condensation in d weather. Window-areas should be as large as practicable, as a large window-a reduces the hours of artificial illumination. If the building is exposed to from another building or buildings of hazardous occupancy, the windows uld be of the Fenestra, Lupton or other equally good, steel construction, ed with wire-glass. The forced circulation of heated air is a very desirable thod of heating mills, and should be used in connection with overhead steam-es.

Floors. As wooden floors are subject to rot, the general floor-construction, possible, should be of concrete or earth or some other non-combustible mate-l. But as the dust rising from floors of such materials injures machinery, as the dripping of oils weakens such floors and seems to make a WOODEN ORING-SURFACE necessary, the following construction is recommended. ken slag or stone, several inches in thickness and thoroughly rolled, is first down, and over this a 4-in layer of tar-concrete. On this is laid a 1-in kness of asphalt, evenly rolled. Over this, 2 or 3-in hemlock planks, bedded hot pitch, are laid and over them a ⅞ or 1⅛-in maple floor, at right-angles the planks.

Column and Beam-Construction. Figs. 14 and 15 show clearly the mode COLUMN AND BEAM-CONSTRUCTION. No beams or other structural timbers uld be painted or varnished until thoroughly seasoned.

The Roofs should be as called for under STANDARD MILL-CONSTRUCTION. SSES in roofs are ordinarily from 8 to 20 ft on centers, the 3-in planks span-g the distance between the trusses as shown in Fig. 14, or resting on PURLINS less than 8 ft on centers, and running longitudinally, as in Fig. 15.

Cornices and Gutters. In Fig. 14, the overhanging OPEN CORNICE is wn, with a drip to the outside and without gutters. Roofs sloping back to de gutters, as shown in Fig. 15, are preferable. Projecting BRICK CORNICES, ch protect the woodwork from outside fires, are shown in Fig. 15. If the ding is exposed to other buildings of hazardous construction and occupancy, APETTED BRICK WALLS and cornices are needed.

oof-Construction. The roof-planks should be at least two bays in length, king joints every 3 ft; or, if purlins are used, the planks should cover at t two spaces between the purlins, and break joints as above. Roof-timbers uld be well anchored to walls in a safe and suitable manner. While the -TOOTH form of roof may be used with this type of building, it may be always necessary or advisable; and the types shown in Figs. 14 and are types common for machine-shops, foundries, and similar buildings, in ch increased head-room is required for traveling cranes. The middle sec-over the crane is often provided with SAW-TOOTH SKYLIGHTS with excellent ts, and the side bays and others are made higher for galleries.

eel Structural Members. In ordinary one-story machine-shops, or in ings of similar nature, where wide spans or trusses are necessary, the of STEEL STRUCTURAL MEMBERS is not objectionable.

Fig. 14. One-story Work-shop. Roof-boards on Trusses

Fig. 18. One-story Work-shop. Roof-boards on Purlins
with thoroughly mopped tarred felt under plank

8. Saw-Tooth Roof-Construction*

The Great Advantages and the increasing use of SAW-TOOTH roof-const tion, and the lack of familiarity with it at many factories, make it desirabl outline important features.

Two Typical Designs are illustrated, Fig. 16, a TEXTILE WEAVE-SHED a good basement for the shafting for driving the looms on the main floor ab thus dispensing with the overhead shafting and belting in the weave-ro and Fig. 17, a design for a light MACHINE-SHOP or FOUNDRY. Other desi using light wooden trusses or reinforced-concrete walls, are applicable.

Roof-Types. It may be well to state here that while light roofs with and 3-in joists and with light boards should never be used, and while the p ciples of SLOW-BURNING OR MILL-CONSTRUCTION, with its heavy timbers, preferred, the increasing difficulty of promptly obtaining yellow-pine lumbe good dimensions, and its increasing cost, often necessitate the use of tru and rather light timbers; but in no case should these timbers be less than in width nor of insufficient depth to carry the load. This, also, is in o that they may be SLOW-BURNING. The roofs in all cases should be constru of planks and have wide bays.

Steel Roof-Trusses. The adaptability of the light forms of STEEL FRAMING TRUSSES, especially when wide spans are needed, often compels t use; and in plants having a safe occupancy, such as that of metal-workers, trusses are not objectionable, providing adequate sprinkler-protection wi good water-supply is available to prevent quick failure of the steel work, to heat from the combustion of the contents of the building or from the bur of the roof. Similar protection is, of course, needed in shops with woo TRUSSES, if disastrous fires are to be prevented; but experience has shown the STEEL-TRUSSED ROOF will fail much more rapidly than one of wood u similar conditions.

Wooden versus Steel Columns. WOODEN POSTS are nearly always a able and should be given preference; but if light STEEL COLUMNS are nece they should be well protected by insulating materials if they are in rooms taining combustibles, as the column is the vital part of the roof-support.

Advantages of Saw-Tooth Roofs may be outlined as follows:

(1) **Uniform Diffusion of Light** throughout the room, thus makin space in it available. With all interior surfaces painted white and with ri glass in the sashes, the DIFFUSION OF LIGHT is almost perfect.

(2) **Better and Cheaper Lighting.** Greater adaptability for ligh large floor-areas in wide buildings with low head-room when compared what is necessary in wide buildings with the ordinary form of monitor-skyli Saw-tooth roofs furnish the true solution of the problem of excluding the d rays of the sun and obtaining the very desirable north light. They resu greater ECONOMY IN LIGHTING, as they lower the fixed charges due to the sm number of hours per day during which artificial light is necessary.

(3) **Better Working-Conditions,** especially in textile-mills, thereby inc ing production and encouraging permanency of employees.

(4) **Special Adaptability to many Industries.** The SAW-TOOTH for especially adapted to weaving and similar processes in textile-factories, to chine-shops, foundries doing light work, and similar processes, such as as

* Taken and adapted by permission from the Boston Manufacturers' Mutual I ance Company's specifications for the construction of saw-tooth roofs.

Fig. 18. Saw-tooth Roof for Textile Weave-shed

Fig. 17. Saw-tooth Roof for Machine-shop

ing and drafting, and to some dye-houses where careful matching of colors is necessary.

Disadvantages of Saw-Tooth Roofs. While the testimony of those who have had experience with SAW-TOOTH ROOFS is almost uniformly favorable, some difficulties have been experienced, practically all of which may be summed up as due to either faulty design or poor workmanship. The difficulties in general are caused by

(1) **Leaks,** due to severe conditions during winter in our northern climates.

(2) **Poor Ventilation.**

(3) **Excessive Heat** when roofs are thin.

(4) **Excessive Condensation** on the underside of roof and glass when the temperature outside is low and there is considerable moisture in the rooms.

Approved Methods of Construction. The following suggestions show how the difficulties mentioned may be obviated if the APPROVED METHODS are applied to special cases by competent engineers or architects. What is good ENGINEERING from the view-point of the manufacturer can also be good FIRE-PROTECTION ENGINEERING, and any design should be adapted to both if the best interests of the manufacturer are to be served:

(1) **Diffused Indirect Sunlight.** As it is desirable to avoid direct sunlight and at the same time obtain an abundance of light, perfectly diffused, the SAW-TEETH should face approximately north and the glass should be inclined to the vertical to take advantage of the brighter light in the upper sky and to prevent cutting off the light by the saw-tooth immediately in front; and, above all, to assure the DIFFUSION OF THE LIGHT over the floor rather than on the under side of the roof-planking.

(2) **Angle of Glass.** For the glass an angle of from 20° to 25° from the vertical and an angle of approximately 90° at the top of the SAW-TOOTH will be about right, the variations depending upon the amount of light required and the latitude. A sharper angle at the top is not needed, as it increases the cost, and makes more roof to be covered and larger spans; more glass, also, is required in proportion, and the light is not as good, as more light from the sky is lost and too much light is thrown on the under side of the roof.

(3) **Glazing-Details.** DOUBLE GLAZING with a space left between the lights of glass is preferred on account of its conducting qualities; but it is not always necessary, except in the more northerly countries. The inside glazing should be done with factory-ribbed glass, set with the ribs vertical and facing it. Shadows cast by trusses are then almost unnoticeable.

(4) **Gutters and Conductors.** CONDENSATION-GUTTERS are needed inside, at the bottom of the sashes, and they should be drained through INSIDE CONDUCTORS and not to the outside under the bottom of the sashes, as these latter admit cold air and are liable to freeze.

(5) **Valleys** between the SAW-TEETH should be flat, from 14 in to 2 ft in width and pitched ½ in per ft towards the conductors, which should be of ample size, and not much over 30 ft apart, and preferably less. The necessary PITCH may be obtained by cross-pieces of varying heights set on top of the trusses, and thus avoiding hollow spaces.

(6) **Prevention of Leaks.** LEAKS, which are common faults, may ordinarily be prevented by a careful design of the gutters, valleys and sashes, and by insisting on good workmanship and materials. The roof-covering of asphalt or pitch should be continuous through the valleys and extend up to the glass.

One form of construction understood to have been very satisfactory is shown Fig. 18 and in connection with it, reference should be made to the papers an discussion on SAW-TOOTH ROOFS in Trans. Am. Soc. M. E., 1907, vol. 28, whi contain much of value.

(7) **Warming and Ventilation.** Experience has demonstrated the adva tage of a combination of DIRECT RADIATION with a FAN sufficient only for VE TILATION and TEMPERING the heat of the room. Heating-pipes should usual be placed overhead and directly under the front of the SAW-TEETH, and run t entire length, and this position assist preventing condens tion. Where there no moving shaftin some forced circu tion is necessary, a it is best obtained a fan, which driv the air from either dry basement or fr outside as may required, and d charges it over hea ing-coils to the st above. In weav and similar roo this is especia necessary and adva tageous in promot the health and co fort of the employe and in making th working-efficien greater. Ventilation and cooling of these large areas with comparatively l stories must not be neglected. Ample vents are needed at the top in form of large metal ventilators with double walls and tight dampers. Th are recommended in place of pivoted or swinging sash, which are apt to l in driving storms, and when open, allow dirt to blow in from the r Good windows are advised in side walls and experience has shown their value

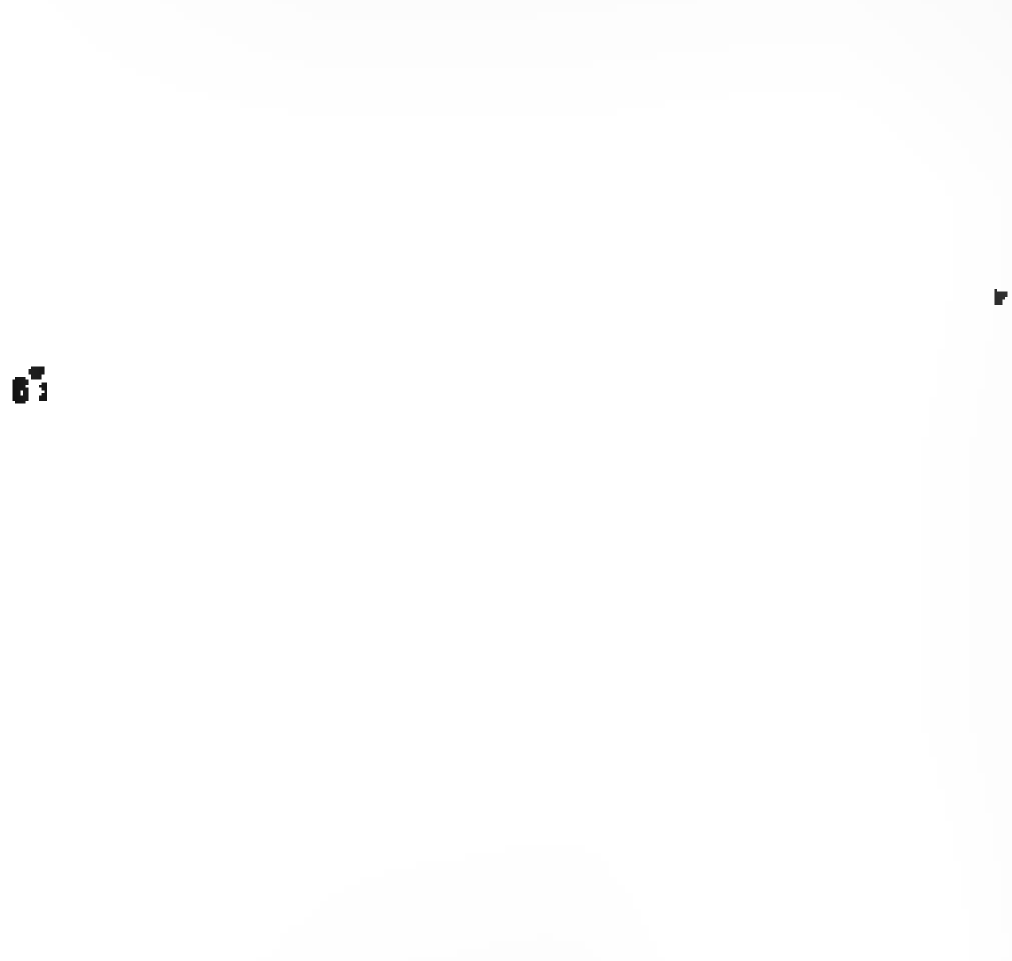


Fig. 18. Detail of Valley of Saw-tooth Roof

(8) **Details of Framing and Construction.** The FRAMING of the SA TEETH may be of timber, steel or reinforced concrete. The design should such as will obstruct the light as little as possible, strong enough to hold snow without sagging, and stiff enough to carry shafting motors, etc., wh they are to be overhead. When wood or steel is used the roof-plank should be 3 in or more in thickness spanning bays from 8 to 10 ft in wid HOLLOW SPACES in roofs should not be permitted. They are very unde able from a fire-standpoint, and any condensation which may take place them during cold weather soon rots both planks and sheathing. SHEATHI even without spaces behind it, is a more or less objectionable feature, as i readily combustible; but if it is used it should be applied directly to the un side of the roof-planks, with only a layer of some insulating material between that there will be no concealed spaces. If 3-in planks are sufficient for a roof, they should be, also, for a SAW-TOOTH roof; and with a good circula of air there should be no trouble, except in wet rooms. In such rooms ther

found to be condensation, whether they are under a roof or under the floor of a room above, unless large quantities of dry air are discharged into them.

(9) **Cost.** SAW-TOOTH ROOFS necessarily cost more than FLAT ROOFS, as there is practically the same amount of roofing as in flat roofs and, in addition, the cost of windows, glazing, flashing, conductors, condensation-gutters for skylights, and a somewhat larger cost for heating. The additional cost of these items does not, however, fairly represent the comparative cost, as there should be considered the total cost of the building compared with that of an ordinary one with sufficiently high stories and with a width narrow enough to give the required light. When this is done the slight additional cost is far outweighed by the advantages gained for work requiring very good light.

9. Mill-Construction as Applied to Warehouses

Cost. Owing to the increasing cost of heavy timbers for wooden construction, to the lower cost of the so-called FIRE-PROOF CONSTRUCTION, and also to the better FIRE-RESISTING qualities of the latter, owners, architects and builders should carefully compare the cost of construction, and also the cost of insurance of the two types, before deciding on the one to be used. The difference in the cost of construction between these two types is so small, that in many localities the lower cost will be in favor of the REINFORCED CONCRETE or other type of FIRE-PROOF CONSTRUCTION. The cost of construction is also in favor of the FIRE-PROOF TYPE, where both long spans and strength are required.

Timber-Spacing for Sprinklers. Warehouses of MILL-CONSTRUCTION should be built so as to allow the best possible distribution of water from AUTOMATIC SPRINKLERS, with the least possible obstructions, and floor-timbers, therefore, should be as few as the floor-loads will allow. There should be no concealed spaces of any kind in the building. To insure the greatest efficiency for sprinkler-systems, it is better to adapt the timber-spacing to suit the sprinklers, rather than to arrange the sprinklers to suit the timber-spacing.

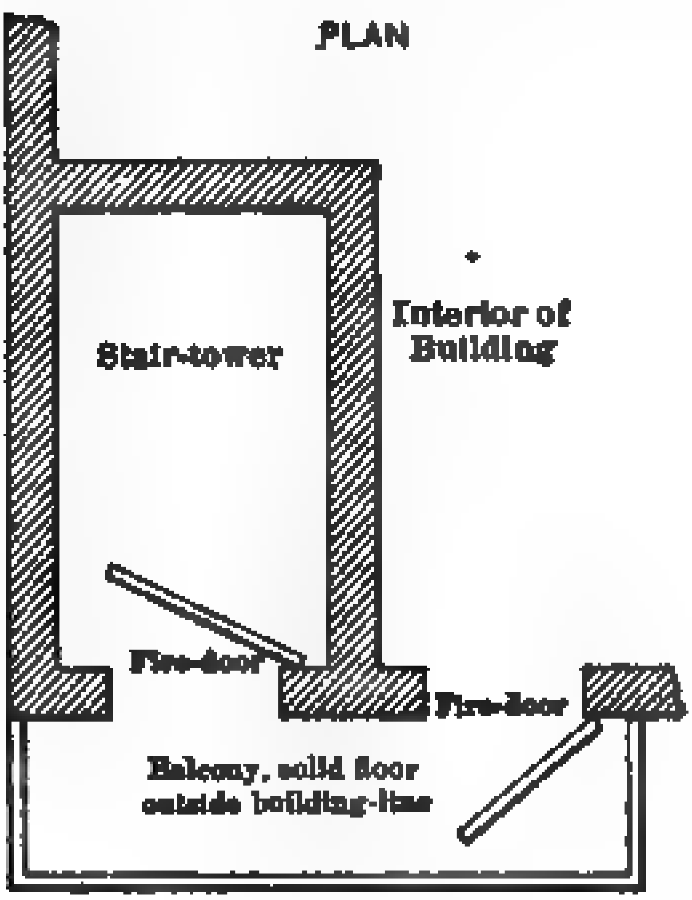
Mill-Construction Adapted to Warehouses. The features of bad construction mentioned under WHAT MILL-CONSTRUCTION IS NOT are as objectionable in warehouses as in factories, while the construction advocated for mills may be used with almost equal advantage in the erection of warehouses. But as the latter are usually erected in the more thickly settled portions of a city, they are more subject to the dangers of a conflagration; and it should be understood that even the best SLOW-BURNING CONSTRUCTION will stand but a short time after a fire has obtained a good headway, the main object of MILL-CONSTRUCTION being to retard the spreading of fire by the use of heavy timbers and the absence of concealed spaces. In applying the principles of MILL-CONSTRUCTION to warehouses, therefore, the general principle of using large timbers placed as far apart as the loads will permit, and of avoiding all concealed spaces, should be constantly kept in mind.

Warehouse-Floors, however, are generally required to sustain heavier loads than are found in woolen and cotton-mills, and hence require heavier construction. While WAREHOUSE-FLOORS are quite often built with transverse girders, 8 or 10 ft apart, the spaces being spanned by flooring from 4 to 6 in thick, a more common method of construction is to use one or more lines of longitudinal girders supporting floor-beams spaced as far apart as possible, preferably not less than 8 ft on centers.

Area and Height. The AREA of buildings of this type should be, preferably, not over 7 500 sq ft, and in no case should it exceed 15 000 sq ft between fire-walls. If buildings of LARGE AREA are required, it is advisable to divide them into

separate sections by fire-walls, thus reducing the liability to one fire, and affording an opportunity of storing hazardous goods in one or more sections, and non-hazardous or less hazardous goods in the remaining sections. Where ground is available, it is better to have a building of LARGE AREA AND LOW HEIGHT divided into fire-sections, than to have a building of LESSER AREA and GREATER HEIGHT, as the former construction affords a more economical handling of goods, and less concentration of values. Buildings of this type should be limited to 65 ft in height, and to six stories, thus discouraging the overloading of floors. Piled goods should be kept at least 18 in away from beams, thus allowing for the distribution of water from the sprinklers.

Walls should be of brick, and not less than 13 in thick in the upper stories, and they should be increased in thickness on the lower floors to take care of additional loads. PARTY WALLS should be increased at least 4 in in thickness, and all walls should be laid in cement mortar, should extend above the roof at least 36 in and be coped with stone, salt-glazed terra-cotta, or similar noncombustible materials. OPENINGS IN DIVISION WALLS should be limited to as few as possible, not over three in each story, they should not exceed 80 sq ft each in area, and should be protected by double, automatic, sliding fire-doors as specified elsewhere. (See Chapter XXIII, page 907.)



Note: Walls of brick or other approved material, built solidly from foundations to at least 36 inches above roof. Stair-treads, etc., of fire-proof material.

Fig. 19. Tower Fire-escape. Outside-balcony Entrance

Openings in Walls. As a protection against fires from surrounding properties, OPENINGS IN OUTER WALLS should be small, limited to as few as possible and protected by standard fire-shutters and doors, or standard wire-glass windows. If the surrounding buildings are of hazardous occupancy or inferior construction, and the distance between the warehouse and the latter but a few feet, shutters are preferable, as wire-glass windows are recommended only where the exposures are moderate. Even though the building is not exposed

fire from other buildings, the protection of WINDOW-OPENINGS may prevent
he spread of fire from story to story through the windows.

Girders and Beams which support the floors and roof should be SINGLE
ECES, not less than 6 in in least dimension, and with a sectional area of not
ss than 72 sq in; while columns should be not less than 8 by 8 in in cross-
ction in the upper story, and should be increased in size in the other stories
take care of any additional loads. The beams and girders should be SELF-
LEASING (Fig. 2), and the floors should be built as outlined under STANDARD
ILL-CONSTRUCTION, page 760, inclined at least 1 in in 20 ft, made as nearly
ater-proof as possible, and scuppered to the outside of the building. These
uppers should be set in brick-work at frequent intervals, of sufficient size to
ry off the maximum amount of water from each floor, and so constructed
at they will prevent the admission of cold air to the building. (See Fig. 11.)

Towers. The floors should be continuous from wall to wall, avoiding holes
r belts, stairways, elevators, etc. All such openings should be enclosed in a
ICK TOWER or in TOWERS extending not less than 36 in above the roof, coped
above, and accessible from each story by means of an outside balcony (Fig. 19).

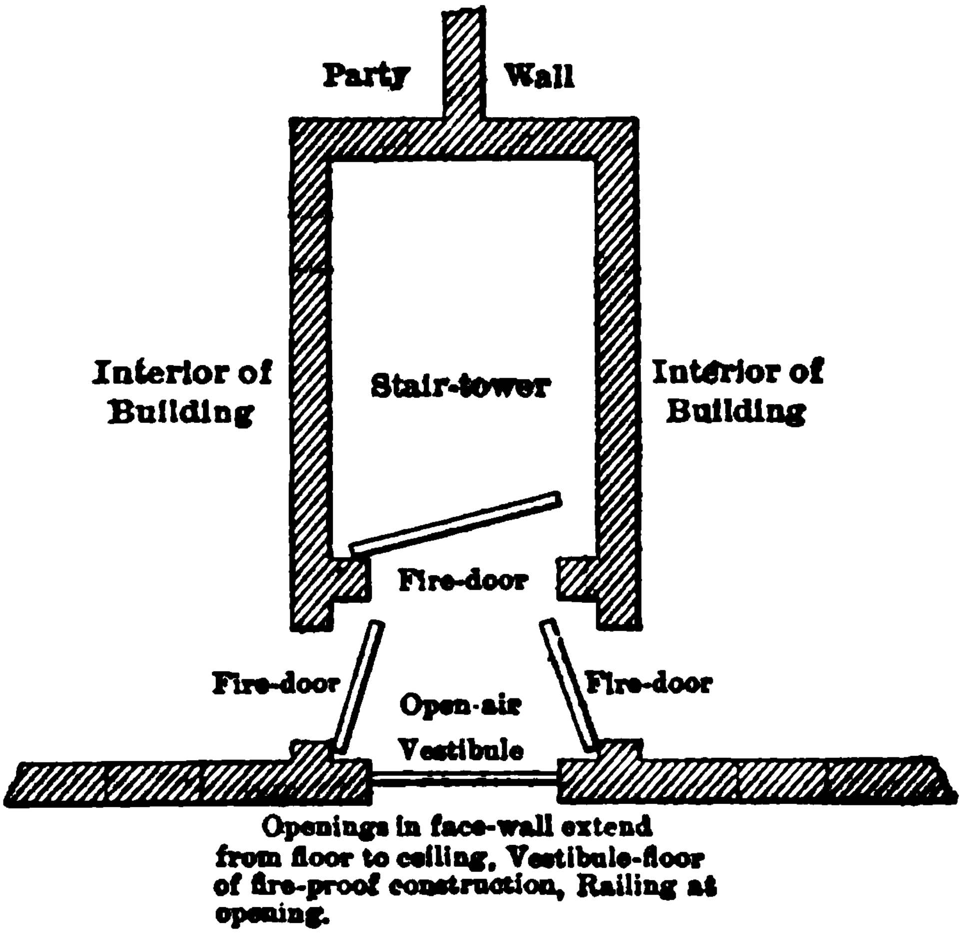


Fig. 20. Tower Fire-escape for Adjoining Buildings

here it is impossible, owing to the location or otherwise, to have these open-
gs on the outside, they should be placed in BRICK TOWERS constructed inside
e building and connecting with an entrance to a fire-proof vestibule, open to
e weather. There should be openings from each story to the vestibule, each
otected by standard fire-doors (Fig. 20).

Gravity-Tanks for Automatic Sprinklers are usually placed on extensions
such towers, and they should be built to carry the additional load imposed.
y access to the roof of the building may be had from a window or windows
ed in the tower, and such opening or openings should be protected by fire-
tters, especially where the tower is elevated a sufficient distance to allow
tank to be placed inside of the tower, thus preventing flames from gaining
ss to the tower and destroying the tank and tank-supports.

Boilers should be, preferably, in a separate building, cut off by stand fire-doors from the warehouse; or, if in the main building, should be located a room of FIRE-PROOF CONSTRUCTION, access to which should be from outs the building only.

Structural Steel Members should never be used in this type of constructi as they will not resist even a moderate fire. If used, they should be protec with fire-proof material. The lintels should be brick arches and not s sections.

10. Steel and Iron Structural Members in Warehouse-Constructi

Metal versus Wooden Standard Members. Owing to the fact tha beam or column of STEEL or WROUGHT IRON when heated will fail by buckl or bending very much sooner than an equivalent beam or post of WOOD, i important that such members be of WOOD, provided that the WOODEN BE have a sectional area of at least 72 sq in, and are not less than 6 in in least dim sion, and that WOODEN COLUMNS have a sectional area of not less than 8 by 8 CAST-IRON COLUMNS, also, will generally fail in fire and water sooner t wooden columns.

Fireproofing Steel Beams and Girders. When STEEL BEAMS and c UMNS are used, fireproofing is necessary to make them as FIRE-RESISTING the floors. Such be and girders may be FI PROOFED as shown in 21. Metal-wire m should be placed shown, and tied to beams and girders metal clips; and insure rigidity during pouring of the conc and to keep the mem alignment, forms sh be used. The conc should be poured be the floors are laid, after the wooden be are in position. A completion, the insulation should be at least 1 in at the edges of flanges, 2 in under the lower flange of the beam and 3 in under the lower fl of the girder. The webs should be filled solid. Where there is little sto of a combustible nature in the building, the beams may be protected as sh in Fig. 22. (See, also, pages 863 to 866.)

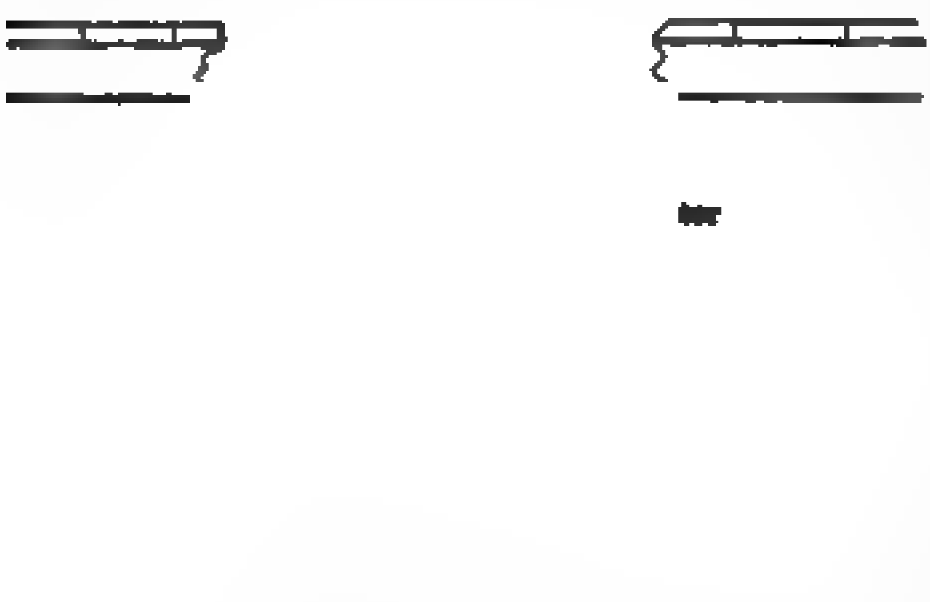


Fig. 21. Fireproofing of Steel Beam with Concrete and Plaster

Fireproofing Metal Columns. COLUMNS, either STEEL, WROUGHT-I or CAST-IRON, should be protected even to a greater extent than girders and bea and should have at least 3 in of concrete at the flanges, at least 1½ in at edges, and be filled solidly against the webs. Fig. 23 shows two columns prote by concrete held by wire mesh on ⅝-in rods, and all securely held to the col by metal clips. Forms should be used and the concrete should be poure the girders and beams are protected. Steel beams, girders and columns difficult to protect, especially at the intersections of steel and wood, and insulating material can best be applied before the floors are laid. The proofing of these members will be of little avail, unless the materials are g

tied to the metal members, and applied by workmen who understand such

Fig. 22. Fireproofing of Steel Beam with Metal-lath and Plaster

Fig. 23. Fireproofing of Steel Columns with Concrete and Plaster

24 illustrates the PROTECTION OF A ROUND COLUMN by reinforced concrete, the concrete is held in position re mesh on metal furring, held sition by metal clips or ties. reproofing should be at least 4 k, and forms should be used in nding the columns. In addition above reinforcements for these as, lateral reinforcement should ed by means of iron rods wound y around mesh, and placed 12 in nters. After the forms are re-, and the wooden floors are laid, lumns and girders should be d with a 1-in thickness of hard, filling all interstices between the woodwork and the insulation. Tile,

Fig. 24. Fireproofing of Cast-iron Column with Concrete and Plaster

owing to the difficulty of properly bonding it, is not as effective as concre but if securely bonded by means of metal, it is quite satisfactory. Fig. illustrates the PROTECTION OF A GIRDER AND A COLUMN by means of tile. The are other equally efficient methods of beam and column-protection, describ in Chapter XXIII. In buildings of warehouse-construction, heavy goods a

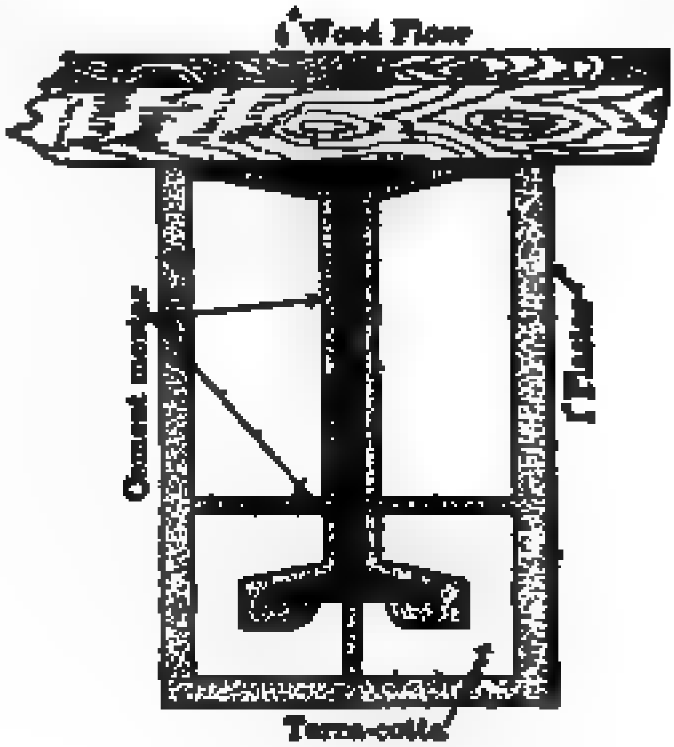


Fig. 25. Fireproofing of Steel Columns and Beam with Tile

handled, and it may be advisable to protect the base of each column with she metal to a height of 36 in above the floor, to prevent any weakening of the fi proofing. (See, also, pages 822 to 827, Figs. 1 to 13.)

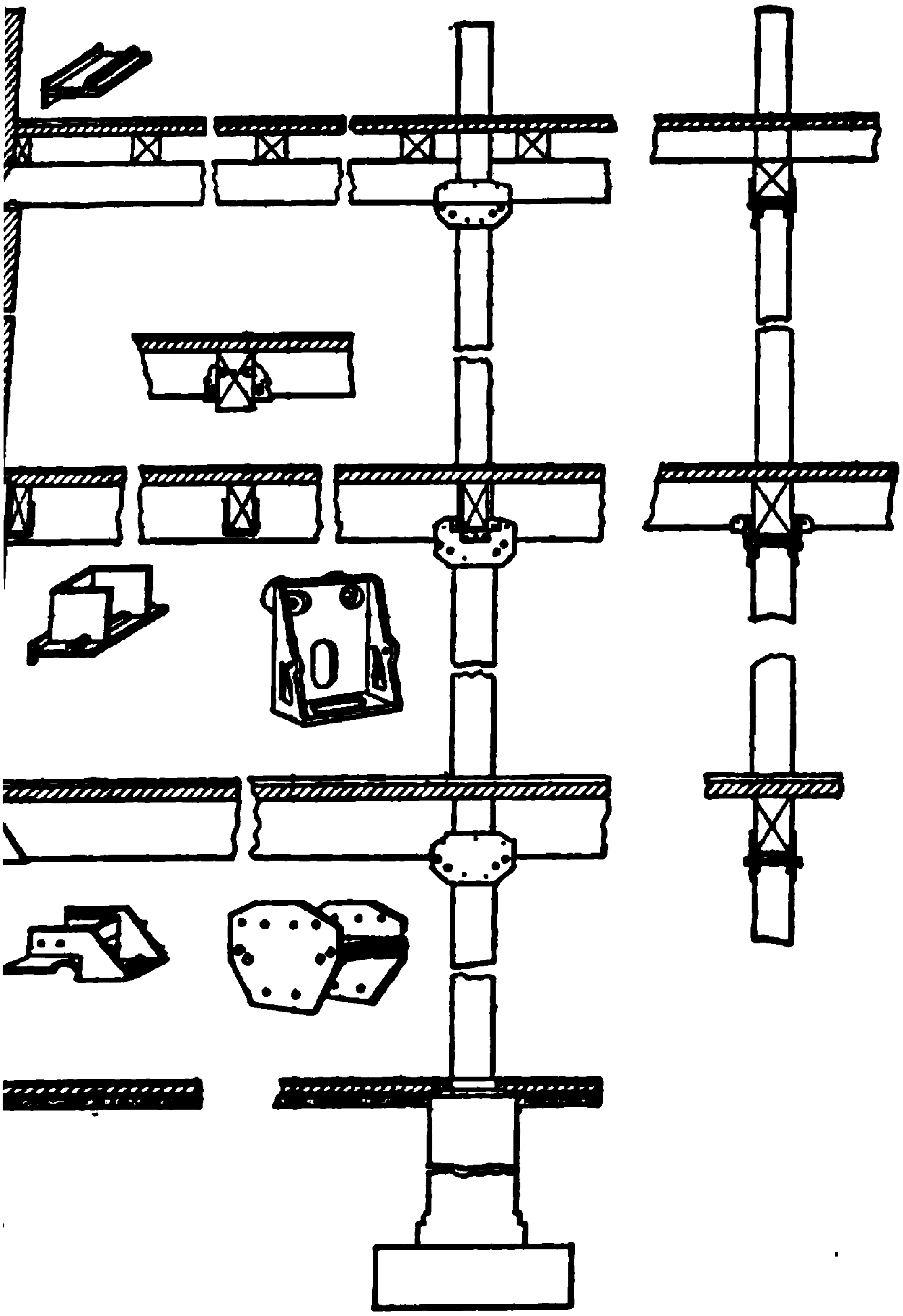
Pipes for Gas, Water, etc., should not be enclosed in column or gird insulation. (See, also, page 827.)

11. Structural Details of Mill-Construction as Applied to Factories and Warehouses

Column, Girder and Joist-Framing. Fig. 26 illustrates the method carrying the girders from the walls, posts, etc., the bottom post resting on a st POST-BASE. The first floor above the basement is shown with longitudi girders only, and heavy mill-flooring set on them. The girders are framed the post in a steel POST-CAP, and are hung clear of the wall in an approv steel WALL-HANGER. The next floor above shows the construction in which joists are framed into the girders by means of JOIST-HANGERS. The frami at the post, also, is done by means of a DUPLEX FOUR-WAY POST-CAP, while girder is built into the wall in a DUPLEX WALL-BOX. The JOIST-HANGERS used singly or opposite each other as required and are bolted to the gird thus tying the building laterally. The upper floor shows the joists resting the girder. This construction, however, does not conform to strict m CONSTRUCTION, as it exposes a larger amount of timber-surface. The gird is shown built into the wall and resting on a WALL-PLATE. This distributes load over the masonry but is not as effective in preventing dry-rot as the wa BOX or WALL-HANGER.

Steel and Malleable-iron Post-Caps and Bases. Fig. 27 illustrates o details of construction which may be used. The bottom post rests on a s POST-BASE. The POST-CAP shown on the bottom post is a DUPLEX FOUR-

st-cap, while the post-cap above it is one of the malleable-iron type, by the National Board of Fire Underwriters. The post-cap shown p, also, is of malleable iron and intended for lighter construction or s which run across the post as shown. The girders in every case are ear of the wall by means of approved wall-hangers and the beams d by the girders in malleable-iron joist-hangers.



. Mill-construction. Column, Girder and Joist-framing

t-Caps and Bases. Fig. 28 illustrates other details of con- owest post rests on a heavy, cast-iron, ribbed post-base. r-girders are carried at the post by means of heavy, cast-iron built into the wall in cast-iron wall-boxes. When cast st-caps it is essential that it be made extra-heavy, as cast ain on account of the uneven shrinkage when cooling, which al stresses and weakens the caps. Flaws, also, may develop

during the manufacture which weaken the caps and greatly impair the saf of the building. An objection to cast iron is its tendency to crack and be during a fire when cold water is thrown on it. The POST-CAPS shown in Fig. are of cast iron for the first and second floors, Duplex steel for the third fl and malleable iron on the top post.

Fig. 27. Mill-construction. Malleable-iron Post-caps and Bases

Duplex, Combination Post-Cap. Fig. 29 illustrates the use of the Dup COMBINATION POST-CAP on the bottom post. This cap is made with a mallea iron lower part and a steel upper part. The POST-CAP shown on the sec post is called the IDEAL POST-CAP and consists of a steel upper part with angles riveted underneath to fit the post. The cap shown on the top

Fig. 29. Mill-construction. Cast-iron Post-caps and Bases

Steel Post–Caps. Fig. 30 illustrates various forms of steel POST-CAPS. The IDEAL POST-CAP is shown on the bottom post and the VAN DORN POST-CAP on the post next above. On the top post the STAR POST-CAP is shown. This has a fin for which the top of the post must be slotted to receive it. Steel JOIST-

HANGERS are shown for the two lower floors. The IDEAL JOIST-HANGER illustrated in the lower floor. It is spiked to the sides and top of the gird The VAN DORN JOIST-HANGER is shown in the second floor, while the old-sty STIRRUP is shown in the top floor. The WALL-HANGERS illustrated are of t approved type.

Fig. 30 Mill-construction. Combination Post-caps, etc.

Framing Steel Beams and Girders. Fig. 31 illustrates the use of I-BE GIRDERS in place of WOODEN GIRDERS and their connections with wooden bea In this kind of construction it is necessary to fireproof the steel beams, as t are more readily affected by heat in case of fire than large wooden timb Intense heat often causes them to collapse and ruin a building. The HAN

...em in the first floor is used where the I beams and wooden beams are of the ...e height. This HANGER provides an extra bearing for the timber and has ...oved very satisfactory. The HANGER shown in the second floor is used when ...is necessary to raise the wooden beam above the lower flange of the steel ...m. This HANGER brings all the load on the lower flange of the I beam and

Fig. 30. Mill-construction. Steel Post-caps, etc.

...vides an anchorage for the wooden beam. It is used singly or in pairs on ...t beam as required, and is bolted through the web of the I beam. This has ... found to be a very economical and efficient construction. In the third ... the wooden beam is shown framed to the I beam by means of a SHELF-...ER. With this form of construction it is necessary to rivet the SHELF-

Fig. 31. Mill-construction. Framing Steel Beams and Girders

ANGLE to the web of the I beam. The upper detail shows the old-fashion STIRRUP passing over the top flange of the I beam and carrying the wood beam. The POST-CAPS shown are the DUPLEX STEEL POST-CAPS which approved by the National Board of Fire Underwriters.

12. Connections of Floor-Beams and Girders

Girder-Hangers and Joist-Hangers. To render the construction, and ticularly the girders, SLOW-BURNING, it is important to have no hollow spaces tween the top of the girders and the flooring, that is, to have the top surface of the floor-beams flush with that of the girders. This, of course, necessitates framing the floor-beams into the girders. For HEAVY CONSTRUCTION the only kind of framing that is permissible is one in

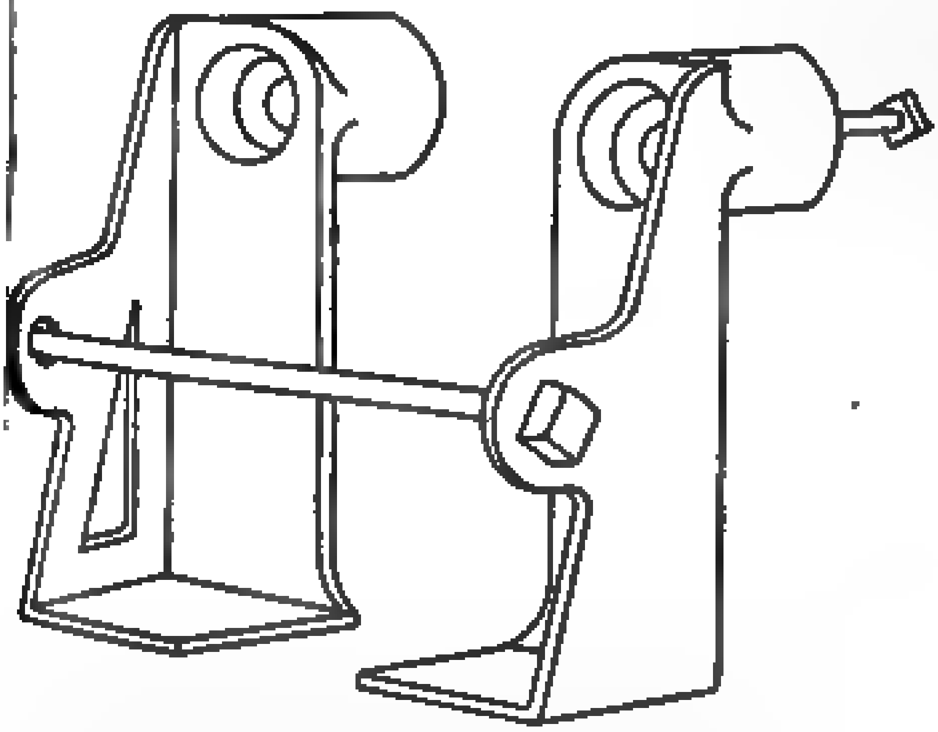


Fig. 32. Duplex Hanger for Heavy Floor-beams

Fig. 33. Framing I Beam and Wooden Beam of Same Depth

hich some kind of JOIST-HANGER is used. The rious kinds of JOIST-HANGERS now in the market ve been illustrated and commented on in the last part of Chapter XXI. hen the floor-beams are 6 by 12 in or larger in cross-section, and the girders e of wood, the author would give the preference to the DUPLEX HANGER own in Fig. 32. (See, also, pages 752 and 753.)

If STEEL-BEAM GIRDERS are used in place of WOODEN GIRDERS, there are veral methods in use for ming the wooden beams. g. 33 shows a steel I beam, d a wooden beam of the me depth framed into it d resting on its lower flange. most cases, however, this s not afford a sufficient ting for the wooden beam. g. 34 shows a SHELF-ANGLE ted to the web of the I m. Whenever this method supporting the beams is d, enough bolts or rivets uld be used to support the d carried by the SHELF-LES. Each ¾-in bolt may considered to support 3 000 n each side of the girder, each ⅞-in bolt, 4 000 lb.

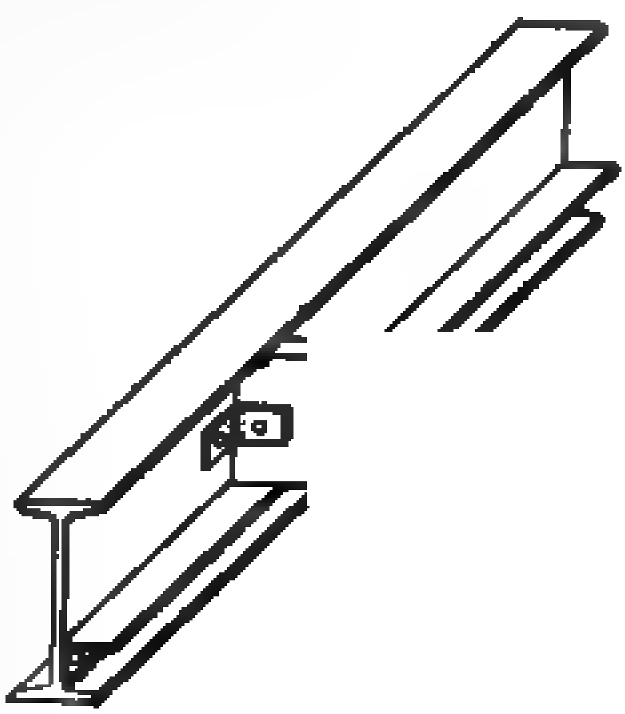


Fig. 34. Wooden Beam Framed to I Beam with Shelf-angle

methods shown in Figs. 35 and 36 are sometimes used, but are open to ction on account of the weakening of the wooden beams when loaded. 37 shows a STIRRUP-TYPE of hanger. This construction permits the

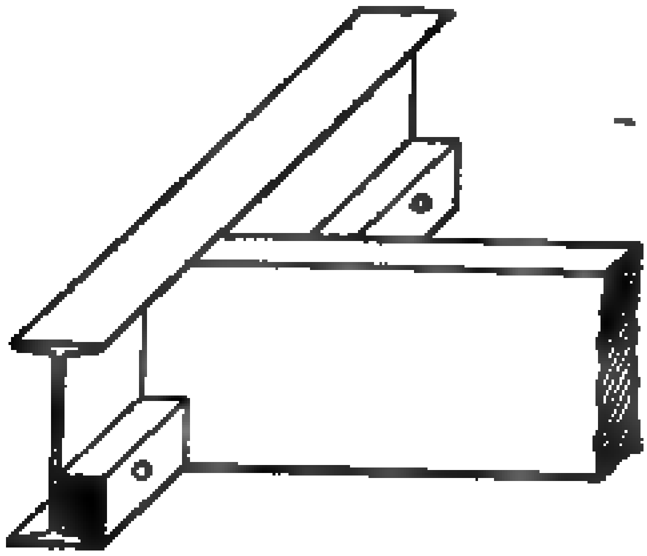


Fig. 35. Wooden Beam Framed to I Beam with Wooden Cleat

Fig. 36. Wooden Beam Framed to I B with Shelf-angle

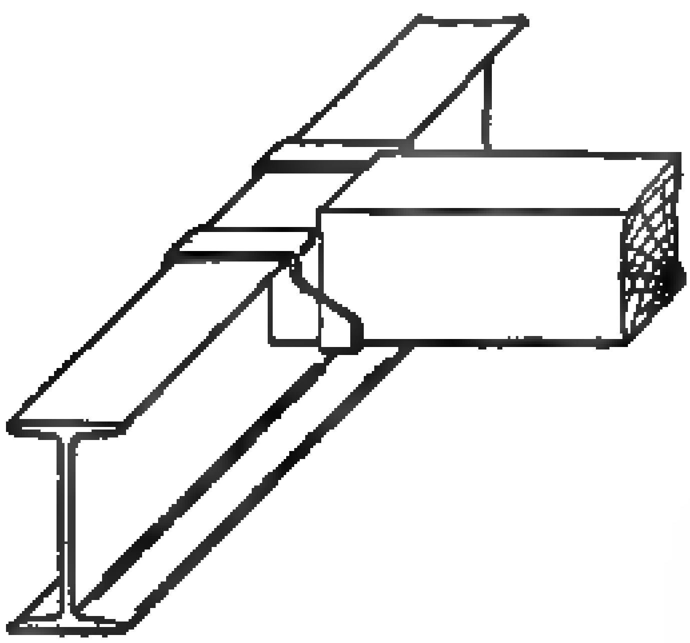


Fig. 37. Wooden Beam Framed to I Beam with Stirrup-hanger

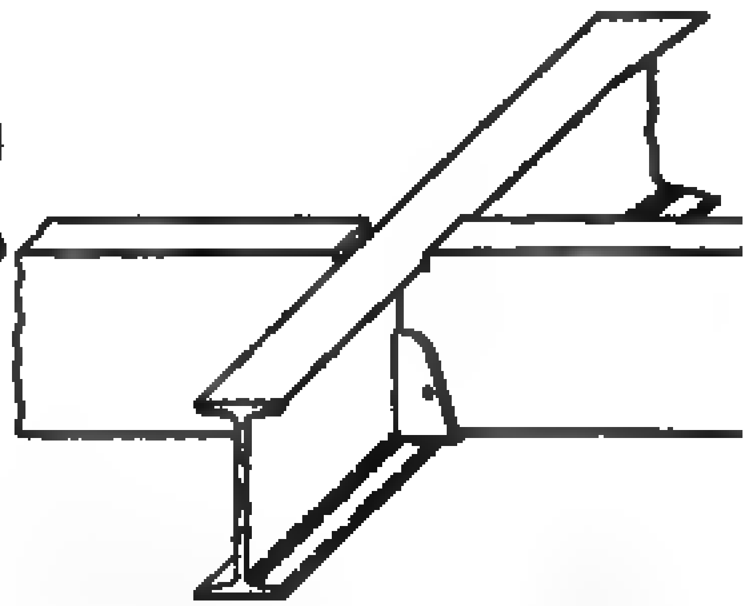


Fig. 38. Wooden Beam Framed to I B with Duplex Hanger

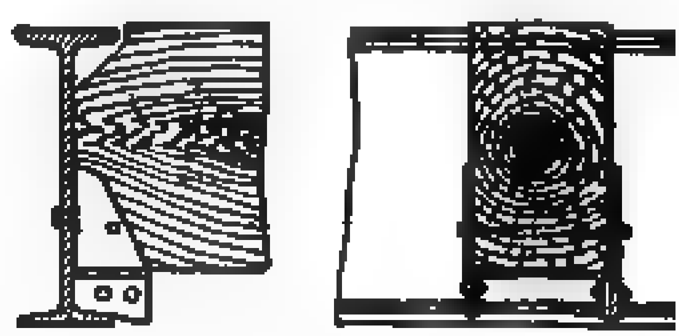
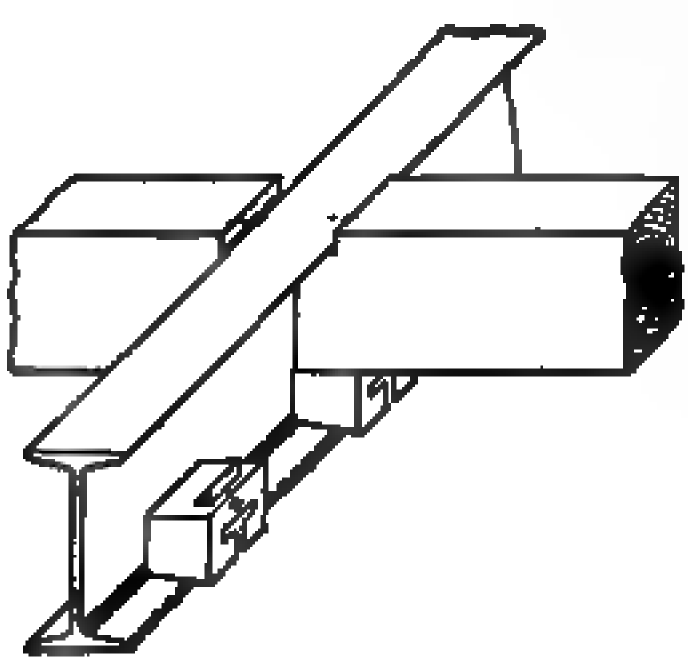


Fig. 39. Wooden Beam Framed to I B with Duplex Shelf-hanger



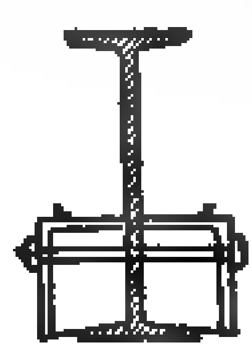


Fig. 40. Wooden Beam Framed to I Beam with Duplex Box-hanger

ing of the wooden beam at any desired height, and has proved satisfactory. hese hangers can be used with any depth of beam or girder, and are furnished y all manufacturers of steel JOIST-HANGERS of the various types, as well as by lacksmiths who can make WROUGHT-IRON STIRRUPS. Fig. 38 shows the DUPLEX-TYPE OF HANGER for framing a wooden beam flush with the lower flange f the I beam. This hanger is attached by means of bolts. Fig. 39 shows

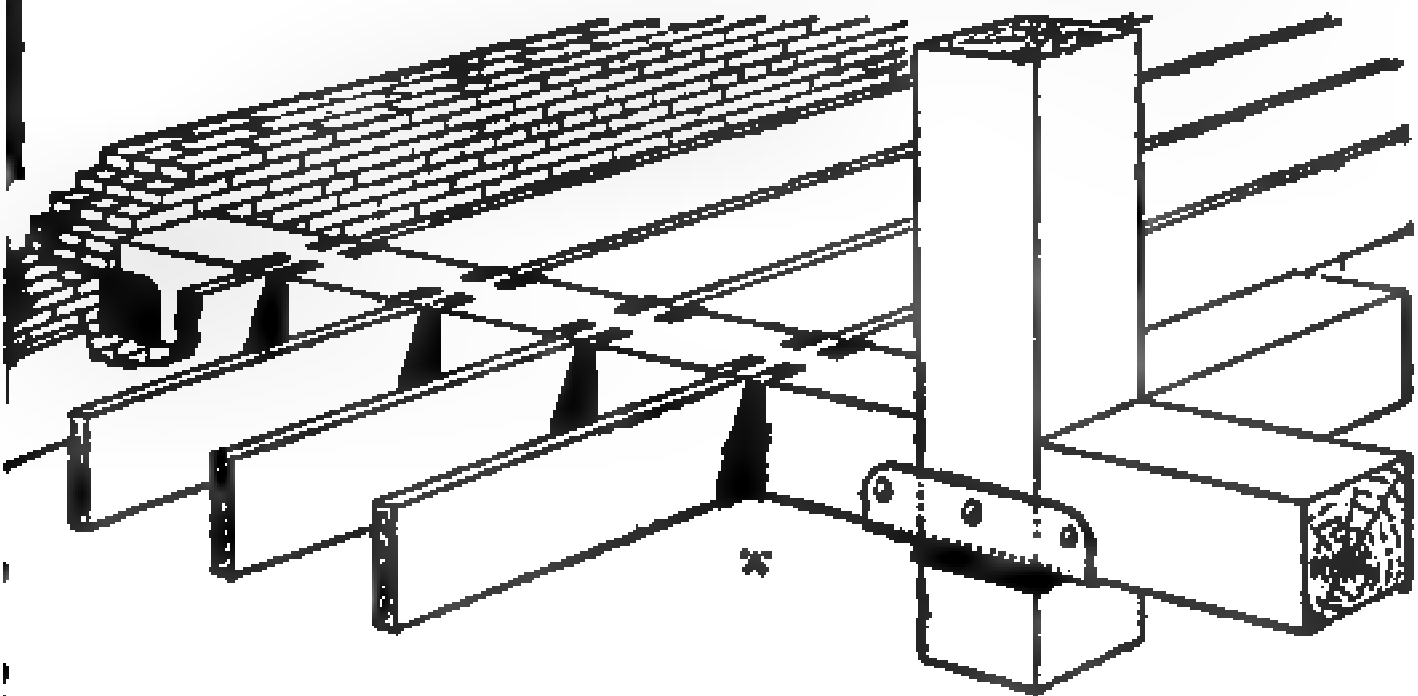


Fig. 41. Floor-framing with Van Dorn Hangers and Post-caps

e same design of HANGER, with the SHELF-CONSTRUCTION used to carry e wooden beams up to 4 in above the lower flange of the I beam. Fig. shows a HANGER for carrying the wooden beams 4 in or more above the wer flange of the I beam.

The HANGERS described in Figs. 38, 39 and 40 are all of the DUPLEX TYPE, d are so constructed that all the load is carried on the lower flange of the

Fig. 42. Floor-framing with Duplex Hangers and Post-caps

m, which is a very satisfactory and ideal construction whenever it is ssary to frame wooden beams into and not rest them on the I beams. The n is a very economical one for framing wooden beams to I beams, as the s for attaching these HANGERS can be punched while the steel is being fabri-

cated, and the HANGERS are attached to the steel beams by means of bolts w the wooden beams are put in place. These HANGERS are provided with or lag-screws for anchoring the wooden beams securely to the steel gir Fig. 41 shows a floor-framing with the VAN DORN STEEL HANGERS. Fig. shows the floor framed with the DUPLEX TYPE OF HANGER and POST-CAP. same principle of construction is applicable to larger wooden beams sp farther apart.

12. Wall-Supports and Anchors for Joists and Girders

Box Anchors, Wall-Hangers, etc. Anchoring. In a warehouse inten to be constructed on the SLOW-BURNING PRINCIPLE, the floor-beams and gir should be anchored and supported by walls in such a way in case the beams burned through, the may fall without inju the walls, and w large timbers are u provision should be m against the possibilit dry rot.

Fig. 43. Early Form of Beam-support in Mill-construction

Box Anchors. method of supporting beams in MILL-CONST TION as originally veloped in the New land mills is show Fig. 43. This full the requirements al mentioned, but it weakened the walls to some extent. The GOETZ CAST-I

Fig. 44. Goetz Box Anchor for Wooden Beams

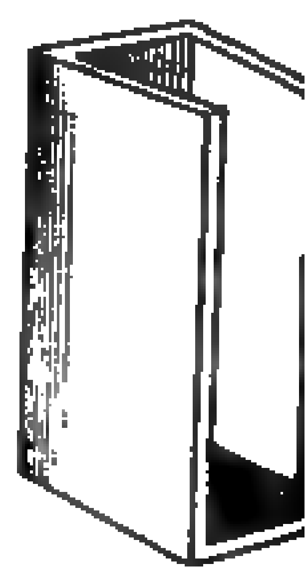


Fig. 45. Goetz Box chor for Wooden B

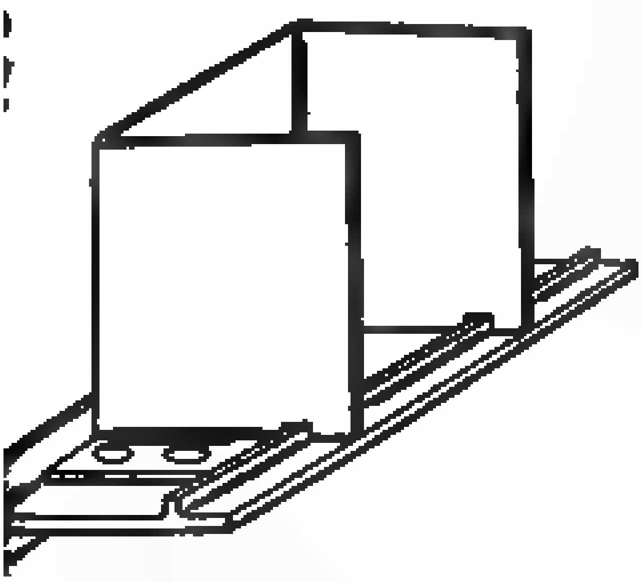
BOX ANCHORS shown in Figs. 44, 45 and 46 and the DUPLEX WALL-BOX sh in Fig. 47 are decided improvements on the anchor shown in Fig. 43, as

nd all the advantages of the latter without weakening the walls, unless the r-beams are very wide. The WALL-BOX as shown in Fig. 47 is made h a malleable-iron bottom plate and a steel box above. It has a rib the plate at the back, which extends up and down, and acts as a secure horage in the brickwork. These WALL-BOXES are made wedge-shape, d it is therefore possible to pull m out of the wall. e more weight re is on the beam, stronger will be bond that holds beam to the box d the box to the ll.

n case of fire or ident, the joists burn through or ak and in falling y can free themves from the anorage and leave wall standing.

Fig. 46. Goetz Box Anchor for Wooden Girders

e wall is not even weakened by the space left in it, because the box ains, and the crushing strength of this CAST-IRON BOX is much greater than t of the wall. No break or breach is made in the wall, and the box that ains, securely held, forms a space for the easy replacement of the wooden m. The box provides a perfect and secure foundation for each beam. from a defective flue cannot ignite a beam-end, because it is protected by a ventilated, CAST-IRON BOX. The WALL-BOXES have air-spaces, also, in the sides, ½ in wide, which permit a circulation of air around the ends of the beams, effectually preventing dry rot. If timber is wet or unseasoned these wall-boxes allow it to dry out after it is put in the building. The average weight of a box like that shown in Fig 45, for 2 by 12-in joists, is 10 lb.



. 47. Duplex Wall-box with Ribbed Plate

Wall-Hangers. Another device for obtaining the same results in a different way is the WALL-HANGER. Figs. 48 and 49 show DUPLEX WALL-GERS for large timbers. The hanger shown in Fig 49 is made of open-hearth l and is extra-heavy. Each of these hangers is provided with a plate which has -in bearing on the wall, and the bearing of the timbers on the hanger is also . For beams not exceeding 10 in in breadth there is probably little choice een the BOX ANCHOR, Fig. 46, and the WALL-HANGERS, Figs. 48 and 49, pt perhaps in the price and appearance. When the WALL-HANGER is used, le is left in the wall, and a saving of 6 in in the length of the beams is ed, which in some cases would be a consideration. For girders 12 by 14 in upwards in cross-section, the author believes that the hanger shown in

Fig. 49 is preferable to the BOX ANCHOR. WALL-HANGERS made from STIRRU should not be used for heavy beams. The use of any one of the hangers

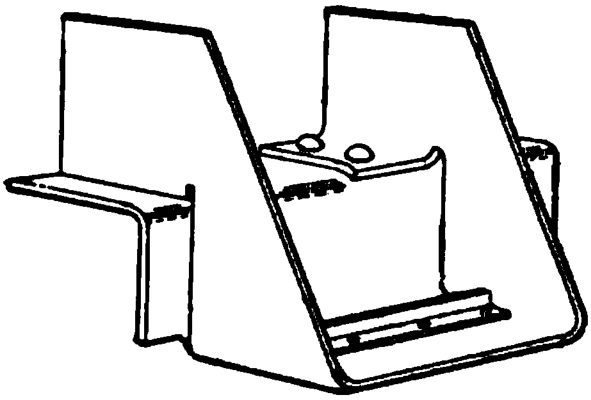


Fig. 48. Duplex Wall-hanger for Large Wooden Girder

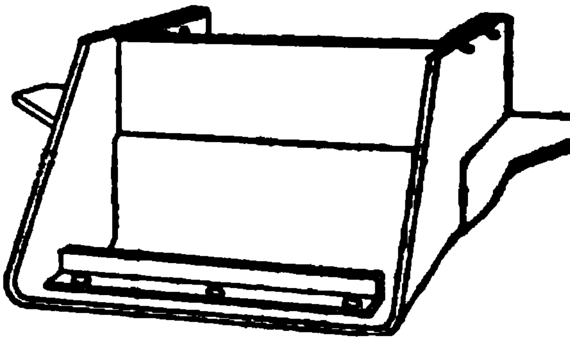


Fig. 49. Duplex Extra-heavy W hanger for Large Wooden Girde

boxes is obviously greatly superior to the ordinary method of anchoring bea or girders to walls, and the use of such hangers will undoubtedly save much which would be caused by the falling of the walls. These are almost invarial

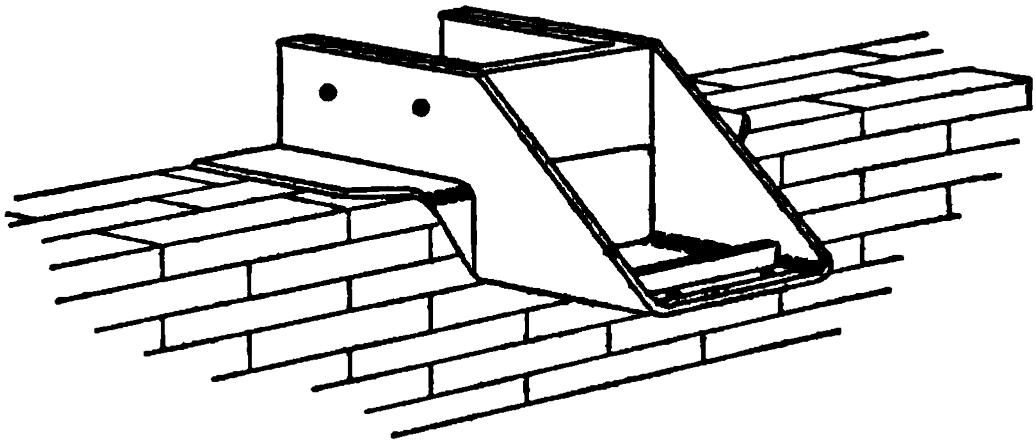


Fig. 50. Application of Wall-hanger to Brick Wall

pulled down by the ORDINARY IRON ANCHORS when the beams fall. Fig. shows the application of a WALL-HANGER.

14. Weakness of Wrought-Iron Stirrups when Exposed to Fire

Stirrups and Fire-Tests. Referring to this subject, Professor J. B. Johns of Washington University, said: "The recent fire-tests of STEEL STIRRUPS brick walls which were made under my supervision in this city (St. Lou show very conclusively that unprotected stirrups are extremely danger These stirrups become red-hot in a few minutes and then rapidly char burn away the ends of the beams; and they also bend down, so that in f twenty to thirty minutes after the fire reaches the stirrups, the beam is drop right out of the twisted steel by the straightening out of this bend or twist."

The Duplex Hangers possess an advantage over STEEL STIRRUPS becau being of malleable iron, they are not as quickly affected by heat, there no twists or bends to straighten, and the bearing in the trimmer or header i a great degree protected by the form of construction. During the severe at Paterson, N. J., February 9, 1902, some DUPLEX WALL-HANGERS were s jected to a most severe test without apparent injury. It is undoubtedly d able that all structural iron should be protected from fire, but it is almost practicable to effectively protect the STIRRUPS used in connection with woo beams without going to a greater expense than the character of the construc warrants.

15. Post and Girder-Connections

[Iron] Cap-Plates, Wooden Bolsters, etc. Whenever a building is con-[str]ucted with wooden posts extending through several stories, each upper post [sh]ould rest on an IRON CAP-PLATE, fitted over the post below, and never on a [gir]der or even on a WOODEN BOLSTER. A BOLSTER would not be objectionable [we]re it not for the fact that the pressure under the post is generally sufficient to [cru]sh the fibers of any kind of wood. Then, too, there is always some settle-[men]t due from shrinkage. As posts are used expressly for the support of beams [and] girders, the IRON CAPS must, of course, extend sufficiently beyond the upper [po]st to afford ample bearing for the end of the girder. This bearing in square [inc]hes should be equal to at least one-half the load on the girder divided by the [saf]e resistance of the wood to crushing across the grain, as given in Table IV, [pag]e 454.

[Ex]ample. A 12 by 14-in yellow pine girder is designated to support a possible [loa]d of 38 000 lb. What bearing should it have at the ends?

[So]lution. The safe resistance given for long-leaf yellow pine to crushing across [th]e grain is 350 lb per sq. in. One-half the load on the girder is 19 000 lb, [an]d hence the bearing area should be 19 000 divided by 350 or about 54 sq in. [A]s the breadth of the beam is 12 in this would require a bearing lengthwise [of] the girder of 4½ in. In no case should the bearing be less than that required [by] the above rule.

16. Form and Material of Post-Caps

Cast-Iron versus Steel Post-Caps. Formerly CAST-IRON POST-CAPS were [use]d for the framing of the girders at the columns and posts. But the uncer-[tain]ty attached to the use of cast iron, and the necessity of extremely heavy [part]s to assure safe construction have led most engineers to specify STEEL POST-[CAP]S, as they are unquestionably the strongest form of construction for fram-[in]g posts and girders. The use of STEEL POST-CAPS is to be recommended, there [bein]g no uncertainty regarding the strength of steel as there is concerning the [stre]ngth of cast iron used for post-cap construction. Internal stresses due to [une]ven cooling may seriously affect the strength of a CAST-IRON CAP, while a [hon]eycombed casting may be used, undetected, and affect the safe carrying [cap]acity; so that failure of the cap may occur even from the vibration due to [the] machinery in the building.

[C]ast-Iron Post-Caps are still used in some localities and a few of the com-[mo]n forms as well as those of STEEL POST-CAPS are shown. Fig. 51 shows a [cap] which is frequently used for light construction. Fig. 52 shows a similar [cap] for a cylindrical post. These caps permit the use of girders wider than the [post]. When the girders and floor-beams are in place, and especially when the [buil]ding is occupied, there is no danger of the girders or posts slipping on [the] plate; in fact it would require a greater force to move them. The girders [shou]ld be tied together longitudinally by IRON STRAPS spiked to their sides. [Man]y persons, however, consider it important in a building of SLOW-BURNING [CON]STRUCTION, to have the posts tied together in vertical lines, and the girders [tie]d in such a way that they will be self-releasing without pulling down [the p]osts. Figs. 53 and 54 show two POST-CAPS which fulfill these requirements. [In] these caps the ends of the girders are not fastened by bolts or spikes, but [are] held in place and tied longitudinally by means of the LUG L on the GOETZ [cap] and by PINS on the DUVINAGE CAP; so that in case the girder is burned to [the b]reaking point, it can fall without pulling on the post. Provision is also [mad]e for bolting the cap to the upper post. The author doubts very much,

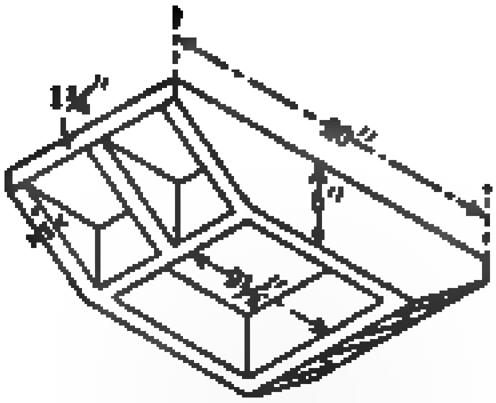


Fig. 51. Cast-iron Post-cap for Square-section Wooden Post

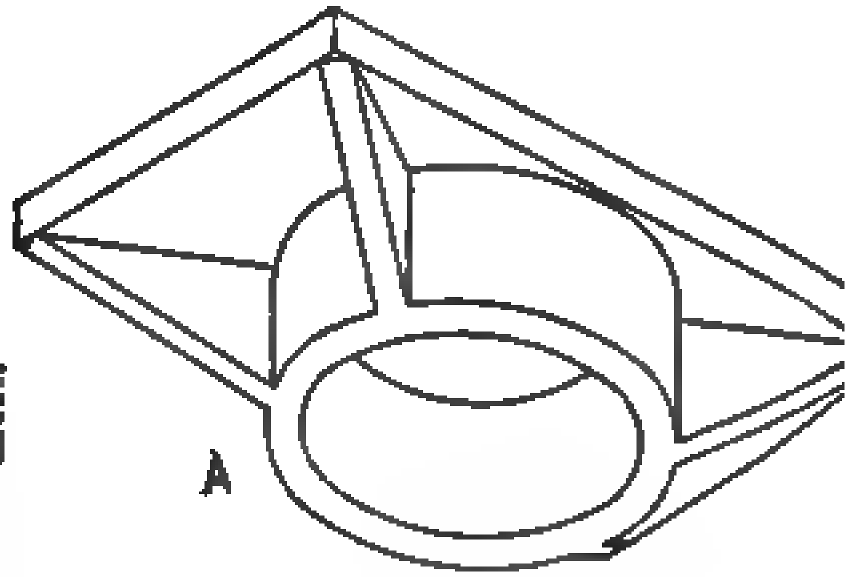


Fig. 52. Cast-iron Post-cap for Cylindrical Wo
Post

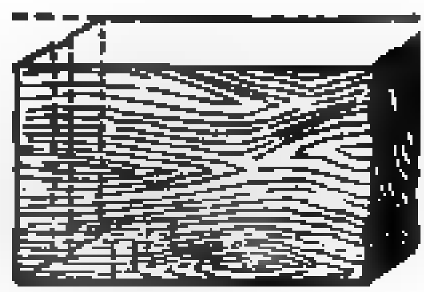


Fig. 53. Cast-iron Duvinage Post-cap with Beam-pins

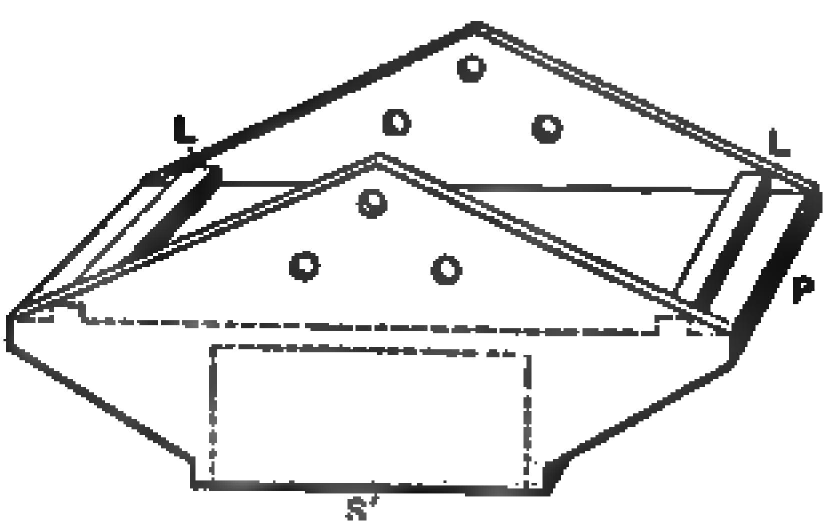


Fig. 54. Cast-iron Goetz Post-cap with Beam-lugs

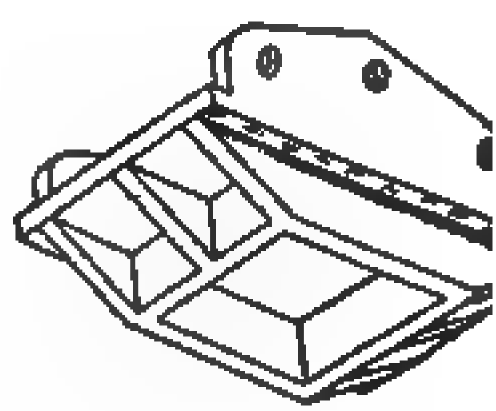


Fig. 55. Cast-iron Post-cap
High Sides

er, if posts bolted together in this way will stand after the girders have as the planking will be likely to pull the posts over, even if they do not as quickly as the beams. Fig. 55 shows another form of CAST CAP with ides, allowing lag-screws to be driven in the holes to tie the girders.

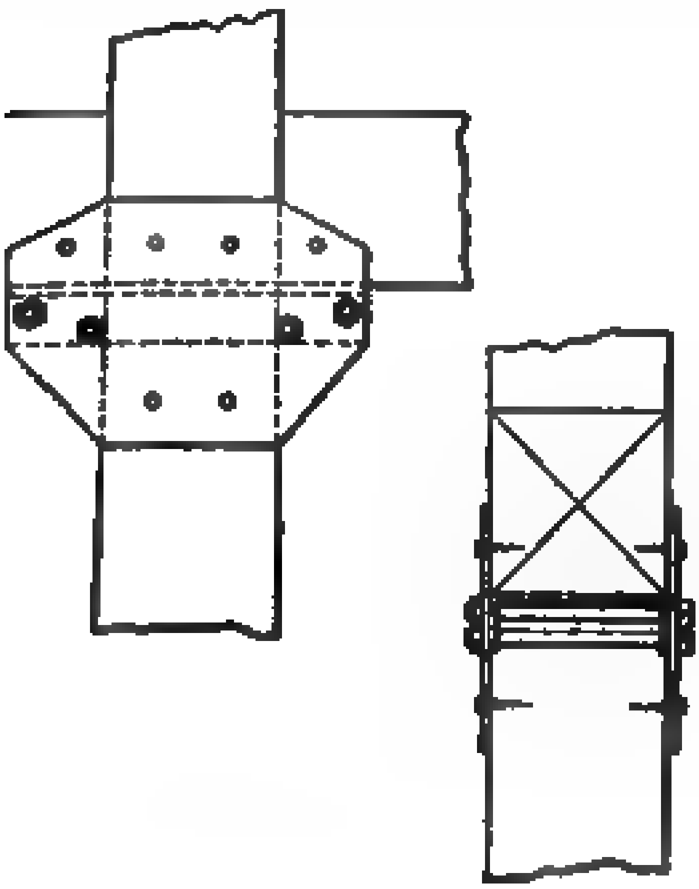
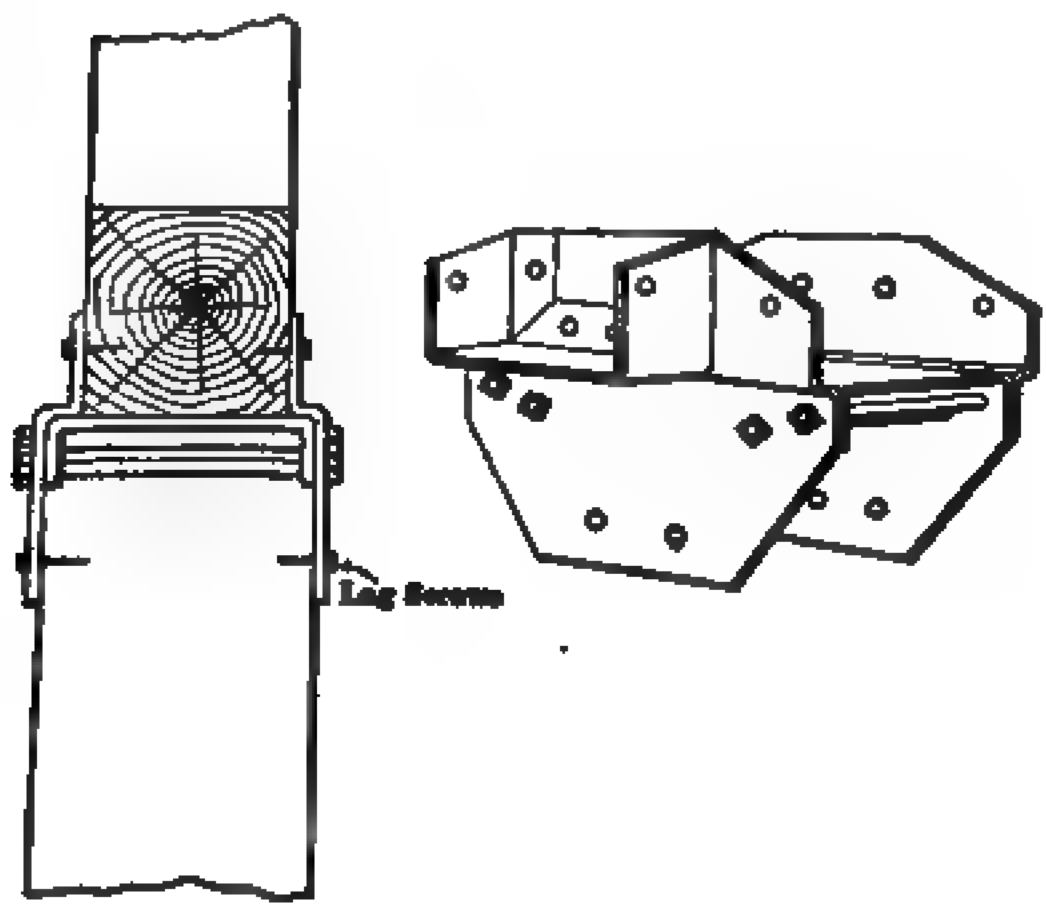


Fig. 56. Steel Post-cap with Side-plates and Brackets



Steel Post-caps for Posts Varying in Section. Second Figure Shows Four-way Beam-construction

Post-Cap, which is approved by the National Board of Fire Under- d bears their label, is shown in Fig. 56. This POST-CAP is made up of plates and heavy steel brackets, all held rigidly together by means avy bolts. The posts and girders are fastened to the cap by means of

Fig. 58. Steel Post-cap. One-way Beam-construction

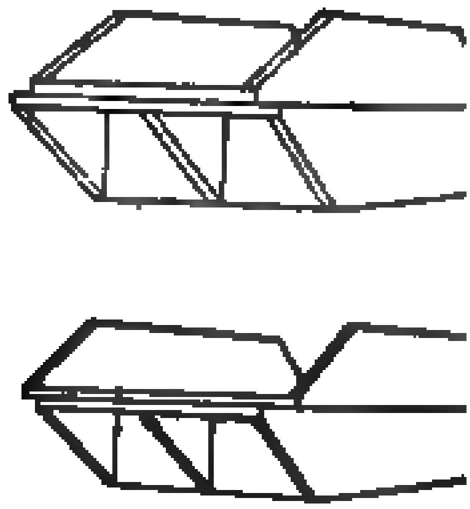


Fig. 59. Malleable-iron Po caps

Fig. 60. Steel Post-cap for Continuous Post

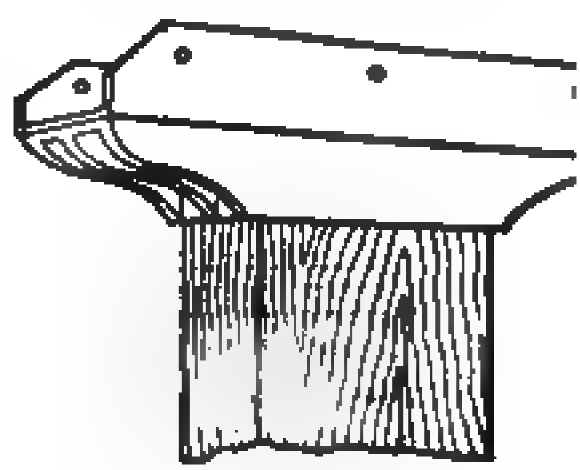


Fig. 61. Malleable-iron Post-cap Steel Top-plate

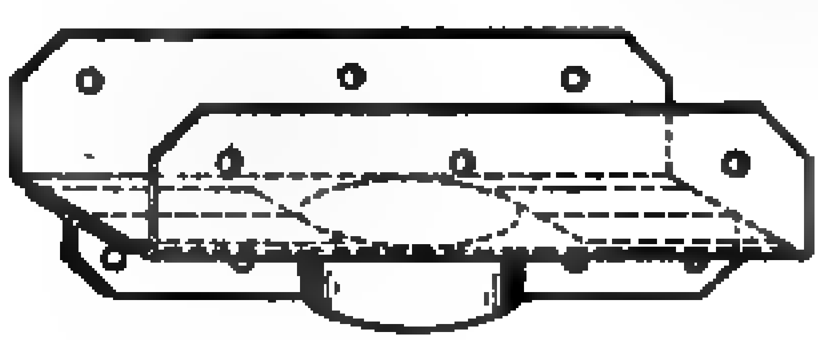


Fig. 62. Steel Post-cap for Cylindrical Wooden Post. Perspective

screws, permitting the girders to release themselves in case of fire. By method the entire construction is tied together vertically and longitudinally. is cap, on account of its simple design, lends itself readily to every form of struction desired.

Various Types of Post-Caps. Fig. 57 illustrates one POST-CAP in which e width of the girder is less than that of the post below, and also another POST-

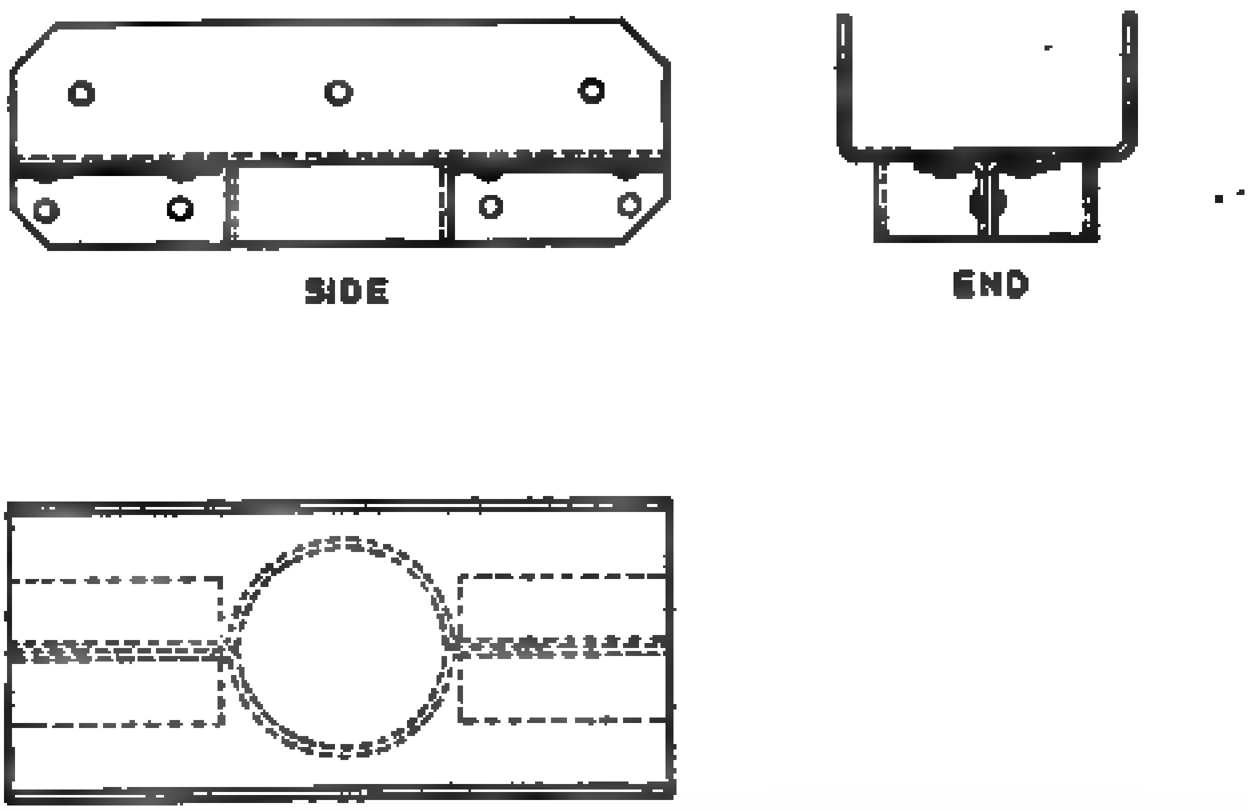


Fig. 63. Steel Post-cap for Cylindrical Wooden Post. Elevations and Plan

p in which the width of the girder is greater than that of the post below. In latter FOUR-WAY BRACKETS are riveted to the side-plates to provide for the R-WAY CONSTRUCTION. Fig. 58 shows a ONE-WAY CONSTRUCTION. Fig 60 ws a POST-CAP which is used when it is required to run a post through two ries. This is what is known as a CON-UOUS POST-CAP. The bracket instead being made clear across the cap is made ut on both sides and fitted into ulders notched into the post, so as make a more rigid construction. Fig. shows two POST-CAPS made of malleable n which are preferable to cast-iron caps they will not break off in case of a fire en cold water comes in contact with m. This danger is present when CAST-N POST-CAPS are used. The cap shown made in two parts so that it will fit ts and girders of different sizes. This , also, is approved by the Board of e Underwriters. Fig. 61 shows a COM-ATION POST-CAP, the upper part of which is made of steel plate, and the r part of malleable iron. Figs. 62 and 63 show STEEL POST-CAPS FOR D POSTS. They are also frequently used for pipe-columns and concrete-l columns. (See, also, Steel-Pipe Columns, page 469 and Lally Columns, s 474 and 477.) Fig. 64 shows a STEEL POST-CAP intended for lighter

Fig. 64. Steel Post-cap for Light Construction

construction. Fig. 65 shows VAN DORN POST-CAPS. Fig. 66 illustra STAR POST-CAP which is made of a bent steel plate with a fin projecting into a slot in the post. Both are approved by the Underwriters. It is ne

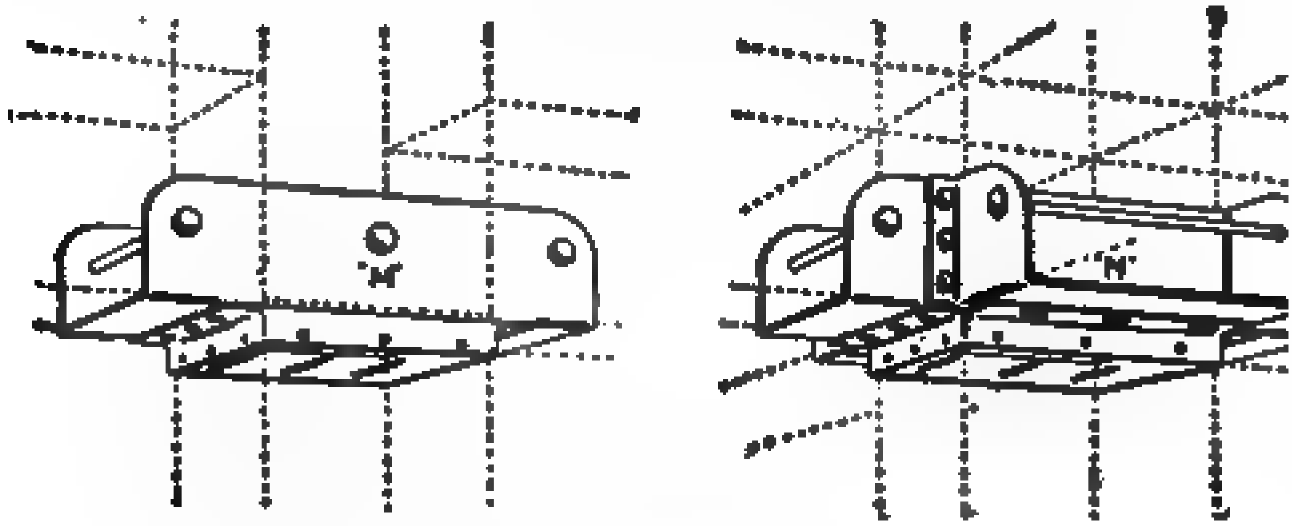


Fig. 65. Van Dorn Steel Post-caps

to slot out the post in order to insert this fin. POST-CAPS which co encircle the top of the post in a socket, to a great measure tend vent the twisting effect of the post, which is so noticeable when th are of wood. There is an o to the use of the FOUR-WAY P when the girders are of wood, the floor-beams that are hung girder drop a distance equal shrinkage in the girder, if th are hung in stirrups, or by this amount if they are DUPLEX HANGERS. The bea ported on the POST-CAP can at all, and consequently the be higher over the beam by the posts, than over mediate beams. In one where deep beams were unevenness in the floor a nearly an inch and was v able. Wherever wooden

Fig. 66. Star Steel Post-cap with Fin

used it is, therefore, a much better construction to support all of beams from the girders, in which case the shrinkage will be unifo steel girders there is no shrinkage, and a beam may be placed o posts with advantage.

17. Roofing-Materials

Warehouse-Roofs are almost always flat and, like floors, should ous from wall to wall, without openings. The occupancy of su calls for little light, and hence skylights and other roof-structures quired.

Dampness and Leaks. Stored goods may be very easily damag and roofs, therefore, should be of such construction that they will p ness, either through leakage or condensation. While roofs are usu flat as possible, the incline should be sufficient to drain readily, a

s should be of sufficient capacity to quickly drain the roof of a maximum ount of water.

ag or Tin are almost exclusively used on buildings of this type, although halt or other mastics are sometimes used with good results.

ag Roofs should be constructed generally as described on pages 1595 1599 and should be not less than 5-ply, with the maximum amount of coating. e flashings and counterflashings should be of copper or heavily-coated best e-plates.

in Roofs should be laid with the best open-hearth, palm-oil-process terne-es, laid on felt or other suitable material which will avoid condensation and as a fire-retardant.

nvas Roofing will stand hard usage, as is shown by its continued use on s of vessels and steamers; but it is not adapted to large buildings.

ovisions for Flooding Roofs. When warehouses are located in congested icts, surrounded by higher buildings, or by buildings of light construction zardous occupancy, their roofs should be so constructed that they may be d during severe fires in such surrounding buildings. This can be accom-d by using good roofing-materials, making high flashings, waterproofing alls above the roof-line, and providing roof-outlets of types that will allow acing of stoppers at the scuppers. (See Fig. 11.)

18. Partitions

n-bearing Partitions. This refers only to those light walls or enclosures separate rooms, etc., and not to those walls which divide the building into s. PARTITIONS, as here defined, bear no floor-loads. Buildings of the URNING TYPE, for occupancies described above, need but few partitions, se should be built of non-inflammable materials, preferably metal lath ster on light, metal studding. All cupboards, closets, lockers, etc., in ing of this type should be of metal, or other equally non-inflammable al.

19. Doors and Shutters

Underwriters' Specifications. Doors and shutters should be built as in the Rules and Requirements of the National Board of Fire Under-for the construction and installation of fire-doors and fire-shutters, as ecifications are accepted by architects and builders as the standard.

Openings should be limited to 80 sq ft, or less, each, and all communica-tween buildings or sections of a building protected with double, auto-ding doors.

20. Fire-Protection

atic Sprinklers, supplied with an ample quantity of water at a good are needed in mills, storehouses, factories, warehouses, etc., where ble goods are made or stored, or where large values are at stake. They ct, be installed in buildings of any type of construction and occupancy, ost effective in buildings of FIRE-PROOF or MILL-CONSTRUCTION.

Standpipes, with outlets in each story, in the basement and on the d be installed at points readily accessible in case of fire, and should icient quantity of good hose attached at each outlet.

ozzles. If a building is badly exposed to other buildings of inferior n or hazardous occupancy, a Monitor-nozzle of large size, located f, is advisable.

Public Water-Supplies. If these are not available, a private fire-se[...] may be advisable.

Competent Supervision. All of the above FIRE-PROTECTION EQUIPM[...] should be installed by men familiar with their operation, and supervised [...] competent FIRE-PROTECTION ENGINEERS, under plans approved by underwr[...] having jurisdiction.

21. Cost* of Mills and Factories Built on the Slow-Burning Princi[...]

Difficulty of Estimating Costs from Tables. The cost of a buildin[...] this type of construction depends upon the cost of material plus the co[...] labor, and as the cost of either varies greatly in different localities the co[...] similarly constructed buildings must also vary. Even if the cost of labor [...] materials does not vary, the cost of buildings of the same area will depend [...] upon the height, floor-loads, distance between bearing-points, design, etc.; [...] it is difficult to deduce a table accurate enough for use in computing even [...] approximate cost of buildings per square foot of floor-areas. One firm [...] architects † states: "Experience has taught us that estimating the cos[...] a building either by the SQUARE-FOOT METHOD or the CUBIC-FOOT MET[...] has proved dangerous and misleading, and it was abandoned by us m[...] years ago except to obtain a general idea of the cost of a building. [...] have found that the only reliable way to approximate the cost of a buil[...] is to block it out and to figure the approximate quantities, which at the ma[...] prices prevailing at the time the building is to be erected, will give the appr[...] mate cost of said building." Owing to the high cost of lumber, a FIRE-PR[...] building will cost but little, if any, more than a building of MILL-CONSTRUCT[...] and owing to this fact it is always advisable to determine the cost of build[...] of both types before deciding upon the type to be used. Buildings of M[...] CONSTRUCTION are becoming obsolete in some localities, and owing to the l[...] rate of insurance on buildings of FIRE-PROOF CONSTRUCTION, those of the l[...] type are much preferred, as in the end they cost less. It is not always sa[...] compare the TOTAL COST OF LABOR with the cost of the LABOR PER DIEM[...] the cheaper labor is often the more expensive in the end, this depending la[...] upon the locality and the conditions imposed. Tables showing the approxi[...] cost of buildings of the MILL-CONSTRUCTION type are computed from the [...] of mill-buildings of light construction (cotton-mills with lateral beams [...] and are not adapted to computing the cost of heavy warehouses or si[...] factory-construction. The figuring of the cost of such buildings from the [...] PER SQUARE FOOT gives, at the very best, only approximate results; and [...] discrepancy of but 1 ct per sq ft will sometimes amount to thousands of do[...] the method is hardly accurate enough to estimate even the approximate c[...]

The Cost of Buildings of Mill-Construction in New England. [...] following eight buildings were designed by Lockwood, Greene and Com[...] of Boston, Mass., who submit data and descriptions of buildings of MILL-[...] STRUCTION with their COST PER SQUARE FOOT. These buildings are, with a [...] exception, situated within a limited area where cost of labor and mat[...] vary but little. The floor-loads vary from 75 to 150 lb per sq ft, and the [...] runs from $0.715 to $1.56 per sq ft. Considering the textile-mills only, [...] average cost is $1.038 per sq ft, while the average cost of all these buildin[...] $1.113 per sq ft.

* These are pre-war prices, but the data are retained for purposes of comparison of [...] tive costs of different types of buildings, or of buildings in different sections of the co[...] For the cost of reinforced-concrete mills, warehouses, etc., see pages 1613 and 1618.

† Farrot & Livaudais, Ltd., New Orleans, La.

A Cotton Spinning-Mill. This mill has an attached picker-house, office d dye-house wings, and was built in Rhode Island in 1911. The following the details of construction: main mill, four stories; size, 263.17 by 131.67 ft; o-story picker-house, 42.67 by 131.67 ft; one-story dye-house 55 by 85.67 brick stair-tank, and other towers; walls of hard bricks; plank and wearing-rs on transverse I-beam framing, supported by cast-iron columns, except five bays, where both transverse and longitudinal framing is used; slag roof-on plank on wooden transverse rafters; floors built for a live-load of 75 lb r sq ft. The cost of the buildings was $0.965 per sq ft.

A Four-Story Cotton-Mill. This mill is without basement. It was built, ther with the fan-room and repair-shop additions in Georgia, in 1910. The wing are the details of construction: mill, four stories; size, 272 by 128 ft; e and repair-shop, one story in height and 122.67 by 36 ft in plan; regular l-construction, that is, brick walls, hard-pine transverse floor-framing, wooden mns and plank floors, except for six bays of the fourth floor which have l I-beam longitudinals in addition to the hard-pine transverse timbers, and sixteen bays of the roof-framing which have both longitudinal and transverse d-pine timbers, these having been found necessary in both cases because of omission of the alternate columns. These buildings have extensive monitors, -tooth skylights, stair-towers, etc. The floors are designed to carry a live d of 75 lb per sq ft. The cost of the building was $0.715 per sq ft.

Cotton-Mill of Irregular Shape. This mill is considerably wider at end than at the other and has a basement at one end. It was built in sachusetts in 1911. The following are the details of its construction: mill, stories; length, 311.67 ft and average width, 75.42 ft; five-story wing, 6 by 40.01 ft, with extensive pent-houses; stair and elevator-towers and ights; brick walls, transverse wooden floor-framing, supported by cast-iron mns and brick walls; conditions at site demanded extensive foundations; dows in fourth and fifth stories of one wall protected by wire-glass in metal es; and floors built for a live load of 75 lb per sq ft. The cost of the buildings $1.172 per sq ft.

One-Story Machine-Shop. This was built near Boston, Mass., in 1910. main building is 200 by 136.375 ft with a connecting wing, 50 by 39.33 ft. s brick walls; longitudinal, steel, I-beam framing; transverse, steel, saw-h skylight framing; plank roof covered with tar and gravel; 20-ft longitu-l and 16-ft transverse bays; steel I-beam columns; 4½-in cement floors pt for three bays which have a 1-in maple overflooring, a 1-in North Caro-pine, intermediate layer, a 3-in kyanized spruce-plank layer, and 4½ in of oncrete; and extensive saw-tooth skylights. The cost of the buildings $1.288 per sq ft.

Building for Manufacturing Automobiles. This building has forge-extensions and was built in Connecticut in 1910. The main building our stories and a basement and is 54 by 151 ft in plan with a one-story sion, 50 by 149 ft, with extensive pent-houses and monitors. The factory-ng has brick walls, transverse yellow-pine framing on heavy wooden col-and on walls, floors of 1-in maple overflooring over 4-in yellow-pine planks, f of 3-in yellow-pine planks covered with tar and gravel, and a 4½-in t, basement-floor. The extension has brick walls; a brick-on-edge floor n a 4-in course of cement concrete on earth; steel roof-trusses, of 47-ft, pan placed 10 ft on centers; tar-and-gravel roof; and extensive monitors. oors are built to carry a live load of 125 lb per sq ft. The cost of the ng was $1.075 per sq ft.

A Two-Story Wooden Box-Factory. This factory has no basement. It was built near Boston, Mass., in 1909. In plan it is 155 by 305 ft and its average height is 32.5 ft. It has brick shafts; transverse wooden framing for the first floor; transverse beams supported by longitudinals for the second floor and roof; wooden columns and plank floors; and wooden monitors. The floors are designed to carry a live load of 150 lb per sq ft. The cost of this building was $0.84 per sq ft.

A One-Story-and-Basement Weave-Shed. This was built near Boston, Mass., in 1909. It is 213 by 244.17 ft in plan, with extensive entrances, towers and saw-tooth skylights. It has brick walls; longitudinal I-beam girders supporting transverse I-beam girders in the first story, resting on brick piers; transverse hard-pine girders supporting longitudinal girders for the saw-tooth skylight-framing; heavy, wooden floors and roof; wooden columns; an earth basement-floor; and foundations on concrete piles. The floors are designed to carry a live load of 100 lb per sq ft. The cost of the buildings, on the one-story basis, was $1.56 per sq ft.

A Two-and-One-half Story Picker-House. There is, also, a two-and-one-half story picker-house and a one-story connecting passage between the two buildings mentioned above for the cotton mill of irregular shape, which were built in Massachusetts in 1911. The picker-house is 64 by 95 ft in plan; the waste-house 21 by 4[illegible]; the covered bridge 10 by 40 ft; and the average height of the building 42.5 ft. The walls are of brick. The picker-house has transverse wooden framing supported by wooden columns and has plank floors. The waste-house wing has transverse, steel I-beam framing and no columns, and concrete-slab floors. The floors are designed to carry a live load of 75 lb per sq ft. The cost of the building, including plumbing, was $1.29 per sq ft.

The Cost of Buildings of Mill-Construction in Philadelphia, Pa., and Vicinity. The following five buildings were designed by Stearns & C[illegible] Philadelphia, Pa., who submit data and descriptions, with the COST PER SQUARE FOOT. These buildings are within a very limited area, being in or within [illegible] miles of Philadelphia, and are of somewhat heavier construction than [illegible] described above, the floor-loads varying from 120 to 150 lb per sq ft and the cost ranging from $0.86 to $1.23 per sq ft. The average floor-load is 132 lb and the average cost $1.02 per sq ft. The two spinning-mills mentioned are designed for average floor-loads of 120 lb and their average cost was [illegible] per sq ft.

A Chocolate-Factory. This was built in Philadelphia, Pa., on open ground. It has an ornamental exterior; walls of Sayer and Fisher bricks with terra-cotta trimmings, and a main building, 83 by 303 ft in plan and two stories in height. One section of the building, 60 ft in length, is three stories high. The story heights are 14 ft from top to top of floors. The floors are designed to carry a live load of 150 lb per sq ft. It has foundations of concrete; heavy plank floors on heavy timber-framing; a slag roof; all stairways and elevators in brick towers; and openings in division walls equipped with fire-doors. The cost of the building, excluding plumbing, heating, electric work, elevators, fire-protection and mechanical equipment, was $0.86.

A Four-Story-and-Basement Chocolate-Factory. This building was erected in Philadelphia, Pa. It is 44 by 130 ft in plan, with average story heights of 13 ft. It was built in a congested part of the city, between other buildings. The cost of underpinning and shoring the adjacent buildings is included in the cost given. It has plain brick walls; slow-burning floor-construction of heavy, wooden timbers, with finished flooring of maple; stairways and elev[illegible]

brick enclosures; and a slag roof. The floors are designed to carry a live d of 150 lb per sq ft. The cost of the buildings including plumbing, but luding heating, electric work, elevators, fire-protection and mechanical ipment, was $1.23 per sq ft.

Spinning-Mill. This building was erected in Philadelphia, Pa., on ground en and easy of access. Its exterior is of brick, without ornamentation. It 4 by 268 ft in plan, three stories in height, the stories throughout being 15 ft n from top to top of floors. The floors throughout are calculated to carry a e load of 120 lb per sq ft. It has walls of brick; a slow-burning floor-con-uction with finished flooring of maple; a slag roof; and stairways and ele-tors in brick enclosures. The cost of the building, excluding plumbing, ting, electric work, elevators, fire-protection and mechanical equipment s $0.93 per sq ft.

Spinning-Mill. This building was erected in Philadelphia, Pa., on ground n and easy of access. Its exterior is a plain brick design. It is 69 by 269 ft plan and three stories in height, the story-heights throughout being 15 ft from top to top of floors. The floors throughout are calculated for a live d of 120 lb per sq ft. It has brick walls with concrete foundations; a slow-ning floor-construction with a finished flooring of maple; a slag roof; all irways and elevators in brick enclosures; and all openings in division walls ipped with fire-doors. The cost of the building excluding the plumbing, ting, electrical work, elevators, fire-protection and mechanical equipment, s $1.07; and the cost of the building including the plumbing, heating, elec-al work, elevators and fire-protection, but excluding the mechanical equip-nt, was $1.34.

Clothing-Factory. This building was erected in Woodbine, N. J., on nd open and easy of access. Its exterior is of brick, without ornamentation. s 45 by 179 ft in plan and three stories in height. The basement is 10 ft in ght, and the other stories 12 ft in height from top to top of floors. The rs are calculated throughout for a live load of 120 lb per sq ft. It has walls rick; slow-burning floors with yellow-pine finished flooring; a slag roof; and tairways and elevators in brick towers. The cost of the building, exclud-heating, electrical work, fire-protection and mechanical equipment, but in-ing freight-elevators and plumbing, was $1.01 per sq ft.

he Cost of Buildings of Mill-Construction in the Middle West. following six buildings were designed by F. G. Mueller, Hamilton, Ohio, submits data and descriptions with the costs of buildings of heavier con-ction. The floor-loads vary from 200 to 300 lb and the cost from $0.62 0.96 per sq ft. The paper-mill at Taylorsville, Ill., is partly of concrete truction, and was built at a cost of $1.30 per sq ft. Exclusive of the last-ed building, the average floor-load is 230 lb and the average cost $0.805 q ft.

Addition to a Paper-Mill. This was built in Dayton, Ohio. It is a story brick building, 116 by 79 ft in plan. The first story is used for paper-ge and the second story as a finishing-room. The first floor is of cement cinder fill; and the second floor of 2¾-in yellow-pine planks with an over-ng of ⅞-in maple, supported by 8 by 14-in beams, 14 by 16-in girders and r 10-in wooden posts. The floors are figured for a live load of 200 lb per . The roof is supported by six steel trusses and 4 by 10-in wooden purlins, covered with 1¾-in sheathing and composition roofing. The foundations f concrete. The cost of the building, exclusive of the plumbing and heat-vas $0.75 per sq ft.

An Addition to a Foundry. This one-story, brick, foundry-building erected in Hamilton, Ohio, is 432 by 63 ft in plan, and has a one-story wi 86 by 46 ft in plan, and a one-story cupola-house, 28½ by 26½ ft in plan. has a wooden floor in the wing only and dirt floors elsewhere. It has conc foundations; a composition roof on 2¼-in sheathing, supported by 12 by 1 girders, 6 by 12-in beams and 6-in cast-iron columns; an elevator in the cup house; and all doors of tin-clad construction. The cost of the building $0.836 per sq ft.

A Paper-Mill. This was built in Monroe, Mich., and is a one-story-a basement brick building, 185 by 87 ft in plan, with an end-wing 234 by 35 It has heavy beam and girder floor-construction, designed to carry a live l of 300 lb per sq ft; concrete foundations and a basement-part, 130 by 87 It is designed for one paper-making machine and four beaters, has a composit roofing and one skylight over the boiler-room. The cost was $0.88 per sq ft

A Paper-Mill. This is an irregular-shaped brick building erected in Ken worth, La., and is 356 by 168 ft in plan. About one-third of it is two sto and the remainder one story in height. It has a heavy wooden, beam, gi and post-construction; a stone foundation on cypress-grillage footings; floors designed to carry heavy paper-making machinery with a live load 250 lb per sq ft. The cost of the building was $0.96 per sq ft.

A Warehouse. This is a one-story-and-basement brick building, erecte Hamilton, Ohio, and is 38 by 50 ft and designed for a live load of 200 lb pe ft. It has a cement floor in the basement; 10 by 14-in girders, 8 by 12-in be and 10 by 10-in posts supporting 3⅝-in flooring; 10 by 14-in girders and 1 round, wooden posts carrying 2¼-in sheathing and composition roofing. cost of the building was $0.62 per sq ft.

A Paper-Mill. This was built in Taylorsville, Ill., and has a main buildi two stories in height and 49 by 130 ft in plan; a one-story part, 138 by 8 in plan; and a one-story wing, 42 by 144 ft in plan. There is a basement u almost the entire building. The foundations are of concrete and there cement floors in the basement. The first floor is of reinforced-beam, gi and slab-construction, designed for a live load of 250 lb per sq ft; the sec floor of mill-construction, supported by cast-iron columns, 14 by 18-in woo girders and 12 by 16-in wooden beams; and most of the roof is supported steel trusses and wooden purlins. The second floor was designed for a live l of 150 lb per sq ft. There are extensive skylights, pent-houses, etc. The of the building was $1.30 per sq ft.

The Cost of Buildings of Mill-Construction in Toronto, Can The building described in the following paragraph was designed by Sproat Rolph, of Toronto, Canada, who submit data of a warehouse-building with floor-openings and windows and other outer wall-openings protected in approved manner, and erected at a cost of $1.12 per sq ft.

A Five-Story-and-Basement Seed-Warehouse. This was built Toronto, Canada, and is 111 by 140 ft 3½ in in plan. The floor-heights ar ft 1 in, and the total height is 66 ft. The floors are built of 2 by 6-in piece pine on edge and the bays measure 12 ft 5 in by 13 ft. The beams are of long- yellow pine, 14 by 18 in in section; the posts of similar material, varying f 8 by 8 in to 16 by 16 in; the walls are of hard, red bricks with gray stone ings; and the sashes and frames are of steel throughout. The building has t elevators in a brick-enclosed shaft and one staircase in a separate brick The floors are designed to carry a live load of 250 lb per sq ft. The c the building, exclusive of the heating and lighting, was $1.12 per sq ft.

…he Cost of Buildings of Mill-Construction in Northwestern Canada. …e following four buildings were designed by J. H. G. Russell, Winnipeg, …ada, and are warehouses of very superior, heavy construction, widely …rated in location, yet varying little in cost. The floor-loads used vary …m 300 to 350 lb, live load, per sq ft and the cost varied from $1.41 to $1.54 … sq ft. The average cost was $1.46 per sq ft.

A Seven-Story-and-Basement Warehouse. This was built in Winnipeg, …nada, and is 50 ft 6 in by 119 ft 9 in in plan. The floors are of 6-in spruce with …in maple overflooring. All floors are on heavy girders and columns; the …vators are in brick shafts; and the walls are of brick, except the first story …nt wall, which is of cut stone. The floors are designed to carry 300 lb per …ft, live load. The cost of the building, exclusive of the heating, elevators, …, was $1.46 per sq ft.

A Three-Story-and-Basement Warehouse. This was built in Winnipeg, …nada, and is 62 ft 6 in by 86 ft 6 in in plan. Heavy fir timbers were used for …ming. It has a 6-in fir-plank solid floor with ⅞-in maple overflooring; …rs and elevators in brick towers; brick walls with the openings in the rear … sides of the building protected. The floors were designed to carry a live …d of 350 lb per sq ft, and the cost of the building, excluding the heating, etc., …s $1.41 per sq ft.

A Warehouse. This is a six-story-and-basement building, erected in Saska…n, Canada, and is 50 by 112 ft in plan. The floors are of 6-in fir, with ⅞-in …ple overflooring, and are supported by heavy fir timbers. The building has …ck walls with a front of pressed bricks and cut-stone trimmings; some of the …nings are protected by wire-glass windows; and the stairs and elevators are in …ck shafts. The floors were built to carry 350 lb per sq ft, live load, and the …lding cost, exclusive of the heating, elevators, etc., $1.44 per sq ft.

A Five-Story-and-Basement Warehouse. This was built in Edmonton, …ada, and is 50 by 137 ft in plan. The floors are of 6-in fir with ⅞-in maple …rflooring. The building has brick walls and the front and one side wall …faced with pressed bricks with stone trimmings. It has the openings in …rear wall protected and the stairs and elevators are in brick shafts. The …s are strong enough for two additional stories and the floors are designed …carry 350 lb, live load, per sq ft. The cost of the building, exclusive of …ting, elevators, etc. was $1.54 per sq ft.

…he Cost of Buildings of Mill-Construction in Vancouver, Canada. … building described in the following paragraph was designed by Dalton & …leigh, Vancouver, Canada, who give data of a warehouse with floors designed …arry an average load of 500 lb per sq ft and costing $1.09 per sq ft. Although …heaviest timbers and the heaviest wall-hangers and beam-hangers were …l, and the floors built of the maximum thickness, the cost was extremely … This no doubt was partly due to the proximity of the timber and the …ities for transporting it by water.

Warehouse for the Storage of Heavy Hardware. This was erected …Vancouver, Canada. The main building has four stories and a basement, …is 85 ft 6 in by 115 ft 6 in in plan. The office-wing has four stories and a …ment and is 60 by 40 ft. There is, also, a four-story and half-length-base…t building, 38 by 120 ft, connecting with the two upper stories of the main …ling by means of a steel bridge 40 ft long. The walls above the basement …f hard-burned brick and the concrete basement walls and floors are treated …hydrolite. The main girders are set 23 ft on centers and vary in section …12 by 16 in to 18 by 24 in and are all one-piece sticks. The posts, set 11 ft

10 in on centers, vary from 12 by 12 in in one piece, to 20 by 38 in, in t pieces. The joists, set 4 ft on centers, vary from 8 by 16 in to 16 by 24 in in piece. The floors are made of 4 by 6-in and 4 by 4-in pieces, laid solid, top flooring made of 2 by 6-in, edge-grain, tongued and grooved pieces, two layers of asbestos between, weighing 10½ ounces per sq ft. All the bers are of fir. There are three brick-enclosed elevators with fire-doors, one elevator in a wooden shaft, built "solid" of 3-in. thick pieces. The of front is of pressed bricks and has plate glass, marble steps and copper t The windows are glazed with wire-glass in metal frames, and there are doors on the outer door-openings. The roof is made of a 6-ply composition a gravel coating. The live load used for the floors varied from 1 000 lb sq ft on the ground floor to 250 lb on the top floor, the average live load b 500 lb per sq ft. The walls and posts were designed to carry two additi stories, with a live load of 225 lb per sq ft. The cost of the building, exclu of the heating and office and warehouse-fixtures, was $1.09 per sq ft.

22. Cost* of Brick Mill-Buildings of Slow-Burning Constructio

Approximate Cost of Brick Mill-Buildings. Mr. C. T. Main† has ma series of diagrams showing the cost in New England, in 1910, PER SQUARE F OF FLOOR SPACE, of BRICK MILL-BUILDINGS of different sizes, from one to stories in height, and of the type known as SLOW-BURNING. The calculat are made for total floor-loads of about 75 lb per sq ft. The figures taken f the diagrams are given on the following page. The costs include ordi foundations and plumbing, but no heating, sprinklers or lighting.

Modifications of the Costs given in Table I: (1) If the soil is poor or conditions of the site are such as to require more than ordinary foundati the cost will be increased.

(2) If the building is to be used for ordinary storage-purposes with low st and no overflooring, the cost will be decreased from about 10% for la low buildings to 25% for small, high ones, about 20% being usually a allowance.

(3) If the building is to be used for manufacturing and is substantially b of wood, the cost will be decreased from about 6% for large, one-story build to 33% for small, high buildings; 15% would usually be a fair allowance.

(4) If the building is to be used for storage and built with low stories and stantially of wood, the cost will be decreased from 13% for large, one-s buildings to 50% for small, high buildings; 30% would usually be a fair al ance.

(5) If the total floor-loads are more than 75 lb per sq ft the cos increased.

(6) For office-buildings, the cost must be increased to cover the exte architectural treatment and the interior finish.

(7) Reinforced-concrete buildings, designed to carry floor-loads of 100 l less per sq ft will cost about 25% more than those of the slow-burning of mill-construction.

* These are pre-war prices, but the data are retained for purposes of compariso relative costs in the analysis made. For the cost of reinforced-concrete mills, wareho etc., see pages 1613 and 1618.

† Engineering News, January 27, 1910.

Table I. Cost of Brick Mill-Buildings per Square Foot of Floor-Area

Length in ft / Width in ft	50	100	150	200	250	300	350	400	500
					One story				
25	$1.90	$1.66	$1.58	$1.54	$1.51	$1.49	$1.48	$1.47	$1.46
50	1.52	1.29	1.21	1.18	1.16	1.15	1.14	1.13	1.13
75	1.41	1.21	1.12	1.08	1.06	1.04	1.03	1.02	1.02
125	1.32	1.09	1.02	0.98	0.96	0.94	0.94	0.93	0.92
					Two stories				
25	2.00	1.62	1.52	1.47	1.44	1.41	1.39	1.38	1.36
50	1.50	1.21	1.13	1.09	1.06	1.05	1.04	1.03	1.02
75	1.34	1.08	1.01	0.97	0.94	0.92	0.92	0.91	0.90
125	1.22	0.97	0.90	0.86	0.84	0.82	0.81	0.80	0.86
					Three stories				
25	1.98	1.57	1.47	1.42	1.39	1.38	1.36	1.35	1.34
50	1.47	1.17	1.07	1.03	1.01	1.00	0.98	0.98	0.98
75	1.30	1.05	0.98	0.94	0.91	0.89	0.88	0.87	0.86
125	1.18	0.93	0.86	0.82	0.80	0.78	0.77	0.76	0.76
					Four stories				
25	2.00	1.61	1.50	1.45	1.42	1.40	1.38	1.37	1.36
50	1.38	1.17	1.10	1.05	1.02	1.00	1.00	0.99	0.98
75	1.32	1.08	0.97	0.93	0.90	0.88	0.88	0.87	0.87
125	1.20	0.93	0.85	0.81	0.78	0.77	0.76	0.75	0.74
					Six stories				
25	2.10	1.72	1.57	1.51	1.48	1.46	1.44	1.43	1.42
50	1.53	1.21	1.12	1.08	1.05	1.04	1.03	1.02	1.02
75	1.35	1.08	0.98	0.94	0.92	0.90	0.89	0.88	0.86
125	1.22	0.96	0.86	0.82	0.79	0.78	0.77	0.76	0.76

e COST PER SQUARE FOOT of a building 100 ft wide is about midway between tha e 75 ft wide and one 125 ft wide; and the cost of a five-story building about mid between the costs of a four-story and a six-story building.

ditional Data for estimating costs of foundation-walls and other wall iven in the following table:

II. Cost of Walls in Brick Mill-Buildings of Slow-Burning Constructio

Number of stories	1	2	3	4	5	6
ndations, including excavations						
st per lin ft:						
Outside walls	$2.00	$2.90	$3.80	$4.70	$5.60	$6.50
Inside walls	1.75	2.25	2.80	3.40	3.90	4.50
k walls						
st per sq ft of surface:						
Outside walls	0.40	0.44	0.47	0.50	0.53	0.57
Inside walls	0.40	0.40	0.40	0.43	0.45	0.47

Columns, including piers and castings, cost about \$15 each.

Assumed Height of Stories: From ground to first floor, 3 ft. Build 25 ft wide, stories 13 ft high; 50 ft wide, 14 ft high; 75 ft wide, 15 ft h 100 ft and 125 ft wide, 16 ft high.

Cost of Floors: 32 cts per sq ft of gross floor-space, not including colun 38 cts, including columns.

Cost of Roof: 25 cts per sq ft, not including columns; 30 cts, incluc columns. Roof to project 18 in on all sides of buildings.

Stairways, including partitions, \$100 each flight. Include two stairways one elevator-tower for buildings up to 150 ft long; two stairways and two ele tor-towers for buildings up to 300 ft long. In buildings over two storie height, three stairways and three elevator-towers for buildings over 300 ft l

Plumbing Fixtures. In buildings of more than two stories figure \$75 each fixture, including the piping and partitions. Allow for two fixtures each floor up to 5 000 sq ft of floor-space, and one fixture for each additi 5 000 sq ft, or fraction thereof, of floor-space.

CHAPTER XXIII

FIREPROOFING OF BUILDINGS

By

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1. Definitions, Areas, Heights, and Costs

Definitions. The term FIRE-PROOF, while now quite well understood by rchitects, is still used in a very broad sense by the public. To be strictly re-proof, a building must be constructed and finished entirely with incom-stible materials, and any of these materials, such as steel or iron, which are riously affected by heat or streams of water must be efficiently protected other materials which are not so affected. This precludes the use of wood, hether exposed or not exposed, also all exposed steel or iron, common glass, d most building stones. It is safe to say that there are very few buildings this country that are absolutely FIRE-PROOF. There are many, however, at could not be destroyed by fire, and in which the salvage would probably mount to from 60 to 80%; and it is the latter class which is generally meant hen the term FIRE-PROOF is used. Incombustible buildings, and buildings of oden construction protected to a greater or less degree from the flames, are metimes advertised as FIRE-PROOF; but such buildings should be considered erely as SLOW-BURNING. It is undoubtedly the duty of every architect to be ll informed concerning the fire-proof qualities of all materials that enter to the construction and finishing of buildings, and to know how to use these aterials to the best advantage. His choice and use of materials is then limited ly by the character of the building and the interests of his clients. It is ended to furnish this information in a concise manner in this chapter. The tional Fire Protection Association recommends the discontinuance of the m FIRE-PROOF, and the use of the term FIRE-RESISTIVE in its stead. The mer term is the one used in the building laws of all the larger cities.

Municipal Definitions. Municipal definitions as to what constitutes FIRE-OOF CONSTRUCTION have a great bearing on the construction of buildings thin their jurisdiction. None is entirely comprehensive and the detailed uirements must be consulted in each case. The Chicago definition is ical of most of them.

Chicago Definition.* "The term FIRE-PROOF CONSTRUCTION shall apply to buildings in which all parts that carry weights or resist strains,† and also all erior walls and all interior walls and all interior partitions and all stairways l all elevator enclosures are made entirely of incombustible materials, and in ich all metallic structural members are protected against the effects of fire coverings of a material which shall be entirely incombustible, and a slow heat ductor, and hereinafter termed FIRE-PROOF MATERIAL. Reinforced concrete

Quoted matter is left in its original form. The editor-in-chief is not responsible for syntax, punctuation, etc.

Stresses are meant.

as defined in this ordinance shall be considered fire-proof construction, wh built as required by Section 550."

When Fire-proof Construction Should be Employed. A building sho be designed, built, and finished to conform to the purpose for which it is to used. A building containing but little inflammable material, and that no great value, need not be as thoroughly fire-proof as one designed for the stor of valuable goods, or for the protection of life in case of fire. The height o building is an important factor in determining whether it should be fire-pr or not. The rate of increase in the difficulty of coping with fire in a build is greater than that of the increase in the height. The area covered by building, also, is important, although in most instances interior division-w may be provided which practically cut up a building into a series of sma buildings. Some of the limitations placed upon non-fire-proof buildings various municipal laws will be found in the following classification and in Tabl page 813.

Limiting Areas for Non-Fire-proof Buildings.

New York City,	7 500 sq ft on an interior lot.
	12 000 sq ft on a corner.
	15 000 sq ft when facing three streets.
Chicago, Ill.,	9 000 sq ft if of ordinary joisted construction.
	12 000 sq ft if of slow-burning construction.
St. Louis, Mo.,	7 500 sq ft.
Boston, Mass.,	10 000 sq ft.
Cleveland, Ohio,	Mill-Construction:
	20 000 sq ft when facing streets on four sides.
	15 000 sq ft when facing streets on three sides.
	12 000 sq ft when facing streets on two sides.
	9 000 sq ft when facing streets on one side.
	5 000 sq ft on any lot when of hazardous occupancy.
Cleveland, Ohio,	Ordinary Construction:
	12 500 sq ft when facing streets on four sides.
	10 000 sq ft when facing streets on three sides.
	7 500 sq ft when facing streets on two sides.
	5 000 sq ft when facing streets on one side.
	2 000 sq ft on any lot when of hazardous occupancy.

Cost of Fire-proof Construction. F. W. Fitzpatrick, found, previou 1903, that fire-proof construction for office-buildings, hotels, etc., adds fr 9 to 13% to the cost of ordinary construction with wooden joists. For st and warehouses the difference will often be less than 5%.* Walter F. Ballin stated (1909) that reinforced-concrete construction cost from 10 to 15% m per square foot of floor-surface than mill-construction and about 25% less th steel-frame and terra-cotta fire-proof construction.† Figures given by J. P. Perry (1911) indicated that reinforced-concrete construction added from 2 to 2 to the cost of mill-construction for commercial buildings, with an averag 6.7% for various localities and all classes of buildings in the United Sta The increase in cost of structural-steel fire-proof construction over reinforc concrete construction averaged 6.4% for fourteen buildings of all classes various localities.‡ More recent comparisons are not available, but it be safely asserted that the increased cost of fire-proof construction over m

* Fireproof, for March, June, and July, 1903.
† Proc. Nat. Fire Prot. Asso., 1909.
‡ Proc. Nat. Asso. Cement Users, 1911.

Table 1. Limiting Heights for Non-Fire-proof Buildings.

Factories	All buildings	Hotels	Schools	Hospitals and asylums	Residence-buildings
Four stories	75 ft	40 ft	• 40 ft	20 ft	75 ft
90 ft	90 ft	Five stories and basement	Three stories and basement	Two stories	Five stories and basement
....	Six stories 85 ft	Four stories	Four stories	Two stories	Four stories
•	90 ft	Four stories above basement	Two stories above basement	Two stories above basement	Four stories above basement
....	75 ft	Five stories	•	....	Five stories
....	60 ft Ordinary construction 80 ft Mill-construction 100 ft Semifire-proof construction				
....	85 ft	45 ft	45 ft	45 ft	70 ft
....	90 ft	....	One story	One story	....
90 ft	90 ft	90 ft	More than 600 persons	90 ft	90 ft
....	72 ft	....	....	....	....
....	55 ft Ordinary construction 84 ft Mill-construction				
....	63 ft	50 ft	40 ft	40 ft	....
....	60 ft	55 ft	....	55 ft	55 ft
....	....	Four stories	Three stories	Three stories	Four stories
....	80 to 90 ft	....	....	Three stories	....
....	Four stories	Three stories	Three stories	Two stories	Four stories
•	59 ft	Three stories	....	....	59 ft

construction and ordinary joisted construction is less than indicated by t figures.

Divisions of the Subject. In constructing fire-proof buildings it is ne sary to consider:

(1) Materials to be used.
(2) Form of construction.
(3) Protecting devices.
(4) Extinguishing appliances.

This general order is followed in the discussion of the subject in this chap

2. Fire-Resistance of Materials

Effect of Heat on Building Materials. All materials of construction more or less injuriously affected by high temperatures. Furthermore, an INC BUSTIBLE material is not necessarily FIRE-RESISTING, as, for instance, s The value of various materials in fire-proof construction is indicated in following paragraphs.

Brickwork. Common brickwork, when of a good quality, will stand expo to severe fire for a considerable length of time. Experience has shown t thick walls are less affected by heat than thin walls, and that hard-bur bricks stand better than soft or underburned bricks. In the Baltimore San Francisco fires, it was demonstrated that for outside walls brick is supe as a fire-proof material to any other material used in wall-construction.

Stone in General. Very few stones successfully stand the action of sev heat, and consequently stone in general should be used very sparingly in f proof buildings, and certain kinds of stone not at all.

Granite will explode and fly to pieces or disintegrate into sand when expo to flames.

Limestone and Marble are usually ruined if not totally destroyed by ordinary fire. They are the least desirable of all stones for use in a fire-p building, and the granites come next.

Sandstone when fine-grained and compact sometimes stands fire with serious injury, but in the case of a severe conflagration it is generally so ba affected that it has to be replaced.

Terra-Cotta is made from clay by mixing it with water into a plastic ma shaping the same into the form desired and baking it at a high temperature kilns. For the usual structural form the shaping is generally done by forc the plastic mass through a special die by means of machinery. Ornamen terra-cotta must generally be shaped by hand.

Ornamental Terra-Cotta. This material, and especially that which has glazed surface, is well adapted for the trimmings of a building that is intend to be fire-proof. It should, however, be made heavy enough to carry both own weight and its share of the wall-load.*

Structural Terra-Cotta. Terra-cotta, as used for floor-arches, column a girder-protection, and for building light, hollow walls, is made of three diffe ent compositions, the material being known as DENSE, POROUS, and SEI POROUS, according to the method of manufacture.

Dense Tiling is made from a variety of clays. Some manufacturers u

* Fire Prevention and Fire Protection, J. K. Freitag.

me or less fire-clay, and combine it with potter's clay, plastic clays, or tough ick-clay. It is very dense and possesses high crushing strength. In outer lls exposed to the weather and required to be light, it is very desirable. Some anufacturers furnish it with a semiglazed surface for the outer walls of build-gs. For such use it has great durability, and effectually stops moisture. In ing dense tilling for fire-proof filling, care should be taken that the tiles are e from cracks, sound, and hard-burned.

Porous and Semiporous Terra-Cotta is made by mixing sawdust with the y, the sawdust being destroyed by the action of the heat, leaving the material t and porous. A small proportion of fire-clay mixed with the plastic clay desirable but not essential. The proportion of sawdust should be from to 35%, according to the toughness of the clay used. Care is required in e process of manufacture to have the work of mixing, drying, and burning oughly done. The burning should be done in down-draught kilns, by a ick process. The product should be compact, tough, and hard, and should g when struck with metal. Poorly-mixed, pressed, or burned tiles, or tiles m short or sandy clays, present a ragged, soft, and crumbly appearance, and e not desirable. When properly made, porous terra-cotta will not crack or ak from unequal heating, or from being suddenly cooled with water when in heated condition. It can be cut with a saw or edge-tools, and nails or screws n be easily driven into it to secure interior finish, slates, tiles, etc. As a ccessful heat-resistant and non-conductor for the protection of other materials, must be ranked very high.

Semiporous Tiling. This material was introduced by those factories which e pure fire-clay in the manufacture of tile, to enable them to compete with e standard porous material. During the process of grinding the clay, about % of ground coal is mixed with it. This coal aids in the burning of the terial and also makes it lighter and more or less porous. Tiling made by s process is admitted to be a much better fire-resistant than the solid or se material. E. V. Johnson says: "personally, I believe that good semi-ous fire-clay tile is fully as efficient as a fire-resisting material as the standard kes of porous terra-cotta."

Strength of Terra-Cotta. (See, also, page 276.) In tests made at Columbia iversity for the building authorities of New York City on terra-cotta blocks en from material delivered in the open market, the following CRUSHING ENGTH was developed:

Table II. Crushing Strength of Terra-Cotta

Description of material	Position of cells in test	Compressive strength, lb per sq in	
		Gross area	Net area
ense tile	Vertical	1 864	4 721
	Horizontal	585	2 613
miporous tile	Vertical	1 027	2 168
	Horizontal	257	1 008

he inequality in strength of the two materials can be overcome by using ker webs and shells for the semiporous or porous material. In the matter

of WEIGHT, porous and semiporous terra-cotta have the advantage over de tile. Dense tiling, when heated and cooled by water, is liable to crack fi the sudden contraction; "blocks with two or more air-spaces are very li to have the outer webs destroyed under this action. Even if not cooled w water, other fires have shown that hard-burned terra-cotta will crack and to pieces under severe heat alone." * The experience of the recent confla tions in Baltimore and San Francisco fully bears out this statement. collapse of the floors of one of the buildings in Baltimore was largely due the weakening of the terra-cotta arches by reason of the breaking off of outer shells. Porous terra-cotta is non-heat-conducting in itself, and the b qualities will usually resist fire and water successfully; but if the product not burned at a sufficiently high temperature to consume all of the sawdust, throwing of cold water upon the heated surfaces will cause an expansion disintegration due to the absorption of the water and its conversion into stear Porous terra-cotta absorbs water freely, and if allowed to freeze when we more or less injured. If the process is permitted to continue, the blocks beco so weakened that they are unsafe for use.

Concrete Blocks and Concrete Tiles.† Numerous forms of building blo and tiles are manufactured of Portland-cement mortar or concrete for use substitutes for brick, stone, and terra-cotta. Concrete blocks are made the DRY PROCESS by tamping a dry-concrete mix into shape in forms, or by WET PROCESS which consists of pouring a semiliquid or SLUSH-MIX into mo and curing the product by air or steam. A third method, known as PRESSURE-PROCESS, is similar to the first, mechanical or hydraulic press being substituted for the tamping. Concrete hollow tile is being made the same uses as terra-cotta tiling, for partitions and floors in general, for enclosure-walls as well as for partitions in residences. For wall-bear purposes, the tiles are usually filled solid for a layer or two where the bea rest upon them. In hollow-block construction, distinction should always made between the strength of the blocks when laid with the core-holes verti and when laid with the core-holes horizontal, as the strength, in the lat position, approximates only one half of what it is in the former. The specifi tions of the American Concrete Institute, 1917, are generally accepted as best practice in the manufacture of concrete blocks. (See, also, Chapter I page 233.)

Concrete Tile. Concrete building tiles have been used for residences Chicago, Ill., Rochester, N. Y., and the suburbs of New York City. T shape and size of the blocks vary with the make of the product. In size shape they resemble terra-cotta tile, though the walls and webs are thic A WET-PROCESS tile was tested by the Bureau of Buildings, New York C in 1911, and showed a COMPRESSIVE STRENGTH in pounds per square in shown in Table III, page 817.

The Trout Concrete Tile Corporation of Flushing, N. Y., has develope method of making hollow tile whereby lightness is combined with streng The Trout tile is made in a hydraulic machine, with a pressure of 1200 lb each sq in of net area of the tile. While this pressure is being applied, particles of concrete are automatically moved about until the voids are fill and the result is a dense, hard product of even quality. This process perm the making of tile with thin walls, thereby reducing the weight to a minimu A tile 8 in high, 8 in wide, and 15¾ in long, with two cells, and with wa

* Fire Prevention and Fire Protection, J. K. Freitag.

† The subject is fully treated in Concrete Engineers' Handbook, by Hool and Johns

Table III. Compressive Strength of Concrete Tile

Dimensions and use			Cells vertical		Cells horizontal	
Height, in	Kind of tile	Number of cells	Gross area, lb per sq in	Net area, lb per sq in	Gross area, lb per sq in	Net area, lb per sq in
8	Wall-tile	2	.....	.....	320	746
10	Wall-tile	2	528	1 510	351	1 228
10	Corner-tile	2	633	1 580	.....	.....
12	Wall-tile	4	510	1 050	360	1 066
8	Wall-tile (Trout*)	2	1 016	2 588	.....	.....

* Trout Tile tested by Bureau of Buildings, New York, in 1914.

nd webs 1 in thick, weighed 91.7 lb. Its compressive strength is given in e last line of Table III.

Concrete. Stone concrete, under the action of heat, is affected much the me way as brickwork The heated surface expands, and as the concrete is very poor conductor, the other surface remains cool and either cracks or uses warping. The heat also affects the strength and texture of the concrete, using a disintegration of the concrete to a depth of about 1 in. Often the rface spalls off with a report. If water is applied after the heat, the surface washed away to the depth of the affected part. These effects vary somewhat ith the stone used in the aggregate. Siliceous gravel has been found by tests d in actual fires to be very destructive to concrete. Granite, on account of e difference between its coefficient of expansion and that of the concrete, is ely to spall. Limestone calcines under the action of heat and is liable to struction for some depth by the water. Trap-rock is a satisfactory material use, from the standpoint of fire-resistance as well as that of strength. If ere is no application of water after the fire and the surface is allowed to cool gradually, the concrete may set again and become hard. It is not well wever, to rely on this. (See, also, Chapter III, page 245, for the effect heat on concrete fireproofing.)

Slag Concrete. Blast-furnace slag has been used as the aggregate in conte, with satisfactory results as to both fire-resistance and strength.* Care st be exercised in the selection of the slag. R. L. Humphrey says that y acid slag should be used and that it must be "dense, tough, and free m sulphur." Sanford Thompson states that the slag must be "air-cooled, shed, screened from dust, and free from foreign material," and that "exceptal care must be used in proportioning, mixing, and placing." †

Cinder Concrete. Cinder concrete, because of its porous character and the ture of its aggregate, makes an excellent fireproofing material. Tests and experience of conflagrations would indicate that it is the best. Care must, wever, be taken in the selection of the cinders. They must be clean furnaceders, free from unburnt coal. When properly selected and proportioned,

For a series of tests and description of materials, see pamphlet issued by the Carnegie l Company, 1911, Furnace Slags in Concrete. See, also, Proc. Am. Soc. for Test. ., 1914. A full discussion of slag concrete is published in the Iron and Coal Trade iew (London), for Nov. 22 and 29, 1918.

Engineering Record, March, 1917.

cinders produce good concrete, but generally a very non-homogeneous mate is obtained, so that its strength is variable and doubtful. If ground machinery before mixing, a better and more reliable concrete is produc In using cinder concrete in floor-construction the working loads are gener determined from load-tests and a high FACTOR OF SAFETY is used. The for practice in New York City was to take one tenth of the BREAKING-LOAD the WORKING LOAD. The building code now prescribes a formula for co puting the strength of cinder-concrete floors, within certain limitations.

Corrosive Action of Cinders. When cinder concrete is used to enc steel, either as a protective covering or as a part of a concrete construction, corrosive effect of cinders must be guarded against. A discussion of t subject will be found in Chapter XXIV, pages 960 and 961.

Mortars, Plasters, and Plaster of Paris. Mortar and plaster must nec sarily enter into the composition of all masonry buildings, whether built brick, stone, or terra-cotta. That ordinary lime mortar, when well made, endure for unlimited periods of time, in dry situations, has been proved actual use. Hydraulic-cement mortars are equally durable in wet or da places. For laying brickwork or tilework in first-class buildings, cement-an sand mortar is preferable to any other; and cement mixed with lime mor gives greater strength than lime and sand alone. Regarding the fire-pr qualities of mortars and plaster compositions there has been much controver the truth of the matter seems to be that all such compositions will withsta the action of heat up to a certain degree, when they are affected in one w or another, depending not only upon the composition but in large measure up their body, and upon the way in which they are used. Lime mortar for wa was formerly considered as the most satisfactory, so far as fire-resistance concerned; but since the improvements in cement-manufacture, cement mort is generally preferred. Lime plaster, applied on wire lath, will withstand a hi degree of heat without injury, but is liable to be washed away in places streams of water. Gypsum plasters, usually termed hard wall-plasters, patent plasters, when applied to brickwork or metal lath, are superior in he resistance to common lime, and the patent plasters will stand the combin effects of fire and water longer than the common mortars.

Plaster of Paris. Compositions of plaster of Paris (gypsum) and brok bricks, wood chips, or sawdust are non-conductors of heat and possess fir resisting properties of considerable importance; and on account of their ligh ness and cheapness, are often used in fire-proof or semi-fire-proof building In France such compositions have been used for generations to form ceilin between beams, and their durability and fireproofing qualities are unquestion in that country. Plaster of Paris compositions when subjected to severe he are softened on the surface, and when water is thrown upon them they wa away to some extent.

Asbestic Plaster. A plaster made by mixing Asbestic with freshly slack lime-putty has been used to some extent in New York City. Asbestic is ma from a serpentine rock, mined near Montreal, Canada, and contains a larg proportion of asbestos. "Claims of great fire-resisting properties are made f this material, as well as resistance to the effects of water during fire; crackin and discoloration due to the percolation of water or acids are also claimed to avoided. The plaster is tough and elastic, and it will receive nails withou chipping or cracking. The weight is said to about half that of ordinar cement mortar." Asbestic was subjected to a severe fire-and-water test i the presence of the officials of the Supervising Architect's office at Washingto D. C., "and the plaster did not crack or drop, but remained intact. All

walls, ceilings, and columns of the appraiser's warehouse in New York City were covered with a coat of Asbestic, from $\frac{1}{8}$ to $\frac{1}{4}$ in thick, applied on the concrete or terra-cotta surfaces. The great objection to the use of this material lies in its slow drying, the time required for a thorough drying out being usually very long." *

Asbestos-Products. Asbestos fiber combined with cement is manufactured in the form of steam-packings, corrugated sheathings, roof-coatings and shingles, wall-boards and building-lumber, insulating sheathing and blocks, asbestos theater-curtains, various forms of preservative and fire-resisting compounds, and substitutes for wall-plaster and stucco. The value of these products lies in their low heat-conductivity and incombustibility.

Asbestos Building Lumber is made in standard sheets, 42 by 48, and 42 by 96 in in size, and varying in thickness from $\frac{1}{8}$ in (about $1\frac{1}{4}$ lb per sq ft in weight) to 1 in (about $10\frac{5}{8}$ lb per sq ft in weight). When seasoned it is harder than ordinary wood, takes nails and screws, and it can be manipulated with heavy tools and machinery such as are used for working iron. It is too hard for ordinary wood-working tools. It is sufficiently elastic to withstand ordinary vibration, expansion, and contraction of surrounding parts, wind-pressure, and blows; and in large pieces, it can be bent around slight curves without splitting.

Asbestos Corrugated Sheathing is corrugated asbestos building-lumber, reinforced with sheet steel of from No. 24 to No. 27 United States gauge, or with open-wire netting. It is applied in the same way that corrugated iron is applied, either nailed to wooden strips bolted to the purlins, or clipped directly to the purlins by clips of hoop-iron or wire. It comes in standard sheets, $27\frac{1}{2}$ in wide and in lengths of 4, 5, 6, 7, 8, and 10 ft.

Asbestos Roofing-Shingles, suitable for wooden-roof construction, possess fire-resisting qualities far superior to wooden shingles. The advantages claimed are their fire-proof qualities, toughness, elasticity, and lightness in weight; ease of manipulation, cutting, sawing, and shaping to fit dormer windows, chimneys, etc.; and their immunity from the corrosive action of salt air. The principal companies manufacturing asbestos building-products are the Johns-Manville Company, New York City; the Keasbey & Mattison Company, Ambler, Pa.; and the Asbestos Manufacturing Company, Lachine, Canada.

Robertson Process Metal consists of steel sheets of from No. 26 to No. 20 United States gauge, enveloped in successive layers of an asphaltic compound containing heavy natural oils, an asphalt-impregnated asbestos-felt, put together under great pressure, and a patented water-proof coating. The sheets are made flat, corrugated, or beaded. It forms an incombustible roofing, siding, sheathing, and interior-finish material. The manufacture of this product is controlled by the H. H. Robertson Company, of Pittsburgh, Pa.

Steel and Wrought Iron. Wrought iron and steel will expand, bend, and twist under a moderate degree of heat. Inasmuch as a temperature of 1 700° F. is not unusual in fires, these materials should not be used in fire-proof construction without proper protection. Fire tests at the Continental Iron Works in 1896 showed that unprotected steel columns under load began to fail when the temperature reached about 1 100° F.† In the Baltimore and San Francisco fires there were many instances of failure in steel columns due to lack of or to insufficient protection.

Cast Iron. "As the result of tests and actual experience in conflagrations it may be stated that unprotected cast iron can stand practically unharmed up

* Freitag.

† See Engineeering News, Aug. 6, 1896.

to temperatures of 1300 or 1500° F. while carrying very heavy loads, even w frequent applications of cold water while the metal is at a red heat."* In tests at the Continental Iron Works, referred to in the preceding paragrap temperature of nearly 1300° F. was reached before the cast-iron columns bega fail. The contents of most mercantile buildings, when burning freely, wo probably generate a heat exceeding at times 2000° F. Consequently, cast- columns, when unprotected, are almost sure to fail in such a fire either by be ing or breaking. No building in which unprotected iron or steel columns used can be considered fire-proof; but in many classes of buildings unprotec cast-iron columns might safely withstand any heat to which they would prob be exposed. From a fire-resisting point of view, when there is no protec covering, cast-iron columns are unquestionably preferable to steel columns.

Fire-proof Wood. To meet the requirements of certain provisions of New York City Building Code, an attempt has been made to produce fire-p wood. The processes for rendering wood fire-proof, in general, consist in pregnating its fibers with certain chemicals. After the fireproofing-proc the lumber should be thoroughly kiln-dried before it is used. The softwo are more easily thoroughly treated than the hardwoods, the resinous wo being particularly difficult to handle.

"The treatment of the wood to render it fire-proof slightly raises the ignit point of the wood. The treated wood is harder to light than the untrea wood, taking two to three times as long to ignite. The amount of wood destro when exposed to the action of a flame is from 5 to 12 per cent greater in case of an untreated wood than in the case of a treated wood. The untre wood furnished more flame than the treated wood. The untreated wood sustain flame longer than the treated wood after the source of heat been removed. From this it can be seen that the fire-proofed wood is likely to ignite and less likely to cause the spread of fire than the untre wood."†

Among the disadvantages of fire-proof wood should be mentioned an incre difficulty in working the wood, and a tendency to dull woodworking-tools rapidly than with untreated wood. Hence an increased cost in the use of proof wood. The salts used in the process of fireproofing being hygrosc tend to keep the woodwork damp. Hardware or other metalwork in co with fire-proofed wood is liable to corrode. The strength of the wood is affected, and in some cases the wood becomes quite brittle. These two mentioned faults can be largely overcome by neutralizing the fireproo solution by a proper mixture of acid and alkaline salts.

The test, known as the timber-test, applied to fire-proof wood in New City, consists in placing a stick of the treated wood, ¾ by 1½ in in cross-se and 8 in in length, for two minutes over a crucible gas-furnace in which a stant temperature of 1700° F. is maintained; then removing the test-p noting the time it continues to flame and glow; and then scraping awa charred wood and determining the percentage of unburned wood. The ditions of acceptance are that, "the flame and glow should disappear w ten to twenty seconds after the removal of the test-piece from the furnace the unburned and uncharred section at the center of the specimen shou not less than 50 to 70 per cent of the original cross-section, depending o variety of wood under test." If the wood has been thoroughly treat splinter of it after having been exposed to flame and withdrawn, will

* Freitag.

† See Insurance Engineering, Vol. IV, page 551; also Professor Norton's B No. 1 to the Boston Manufacturers' Mutual Fire Insurance Company.

glow or flame. Other tests have been suggested and used but need not lescribed here.

Fire-Glass. The introduction of this material has made it possible to se fire-protection in many cases, without the necessity of disfigurement due re-shutters. Wire-glass is either RIBBED, ROUGH, MAZE, COBWEB, or POLISHED TE, with wire embedded in its center during the process of manufacture. The temperature at which the wire is embedded in the glass insures ad- on between the metallic netting and the glass, and the two materials become and inseparable, so that if the glass is broken by shock, by intense heat, or other cause, it remains intact." It is this property of remaining intact gives it its fire-retarding qualities. Although fire and water may cause ks to spread throughout the glass, the wire holds the pieces so firmly that es cannot pass through it. Many severe tests during actual fires have tively demonstrated the truth of the above claim. For warehouses and ries the RIBBED OR MAZE glass is generally preferable; but for offices, or ever clear transparent glass is desired, the POLISHED PLATE is nearly if quite as acceptable as the same glass without the wire, the effect being the as that obtained by looking through a window with a screen on the out- Where FIRE-RESISTANCE is the desired feature, the following requirements ld be satisfied. The thickness of the plate at the thinnest part should t less than ¼ in, and the plane of the wire mesh should be midway between wo surfaces of the glass. No wire should be smaller than No. 24 Brown arpe gauge. The unsupported surface of the glass should not exceed q in in any case and should be contained in a metal frame not larger than 9 ft between supports. The chief manufacturers of wire-glass in this coun- are the Pennsylvania Wire Glass Company, Philadelphia, Pa; the Mis- pi Wire Glass Company, New York; the Western Glass Company, Streator, and the Highland Glass Company, Washington, Pa. As now manufactured e continuous process, it is rolled in lengths up to about 10 ft and in thick- s up to ½ in.

sm Glass. Prisms installed for the purposes of increased light are usu- not contained in frames which are designed to withstand severe heat. dimensions of the unsupported electro-glazed panel should not exceed in either direction. The polished plate in prism-glass units should not d 4 in in either direction, with a minimum thickness of 3/16 in. In Report of the Insurance Engineering Experiment Station, C. L. Norton describes es of comparative fire-tests on electro-glazed Luxfer prisms, 0.35 in thick in square; electro-glazed plate, ¼ in thick and 4 in square; and ¼-in wire- The results of these tests indicate that the three materials, in sheets up by 30 in, are of equal value in FIRE-RESISTANT PROPERTIES and remain ctive operation up to the time when the temperature of melting glass is d. (See, also, page 1578.)

-proof Paint. Numerous so-called FIRE-PROOF PAINTS have been in- ed in recent years. When applied to woodwork they provide a more or fective protection against fire and may, for this reason, prevent the of fire. The following regulations regarding fire-proof paint were in the annual report of the Manhattan Bureau of Buildings, New York, 4.

The term FIRE-PROOF PAINT shall be understood to mean any prepara- ed to cover the surfaces of wood or other materials for the purpose of ing the same against ignition.

No fire-proof paint will be considered satisfactory unless it so protects od or other material to which it is applied that the same will not flame

or glow after having been subjected to the flame of a gasoline torch for minutes.

"(3) Before applying fire-proof paint to any material the surfaces must cleaned.

"(4) Application of fire-proof paint must be repeated whenever it is fo that the material to which it is applied is no longer protected to fulfill Spe cation No. 2."

8. Column-Protection

Girder and Column-Protection. As the columns and girders of a build form the BACK-BONE of the structure, it is of vital importance that they be thoroughly protected from heat. As a rule, the manner of protecting th structural elements depends quite largely upon the floor-system adop Where concrete is used for the floor-construction it is generally also emplo for incasing the columns and girders; where hollow tile is used in the floors, same material is almost invariably employed for protecting the steel fra The methods used for protecting girders are described in Subdivision 4 of chapter. (See, also, pages 780 to 782.)

Necessity for Column-Protection. It is now generally recognized that and steel columns should be incased with some material that will thoroughly tect the metal against fire. In 1896 a committee of the American Society Mechanical Engineers, in conjunction with representatives from other organi tions, made a series of fire-tests on full-sized unprotected cast-iron columns steel columns, loaded to their figured safe capacities. These tests showed the steel columns failed at an average temperature of 1 150° F., and the c iron columns at an average temperature of 1 300° F., the failure setting in a an exposure to the fire of from 23 minutes to 1 hour and 20 minutes, or average duration of about 50 minutes. In order to determine the value several materials as satisfactory protective coverings, the Bureau of Buildi

Table IV. Tests of Protective Coverings

Materials under test	Temp. on face of protective material, degrees Fahr.	Temperature of plate at back of protective material, degrees Fahr.		
		Before heating	After heating for 2 hr	Heat trans-mission
Terra-cotta: dense, hollow, 2 in thick..	1 700	75	223	148
Terra-cotta: semiporous, solid, 2 in thick	1 700	73	244	171
Plaster of Paris and shavings, 2 in thick	1 700	69	159	90
Plaster of Paris and asbestos, 2 in thick.	1 700	70	163	93
Plaster of Paris, wood fibers, and infusorial earth, 2 in thick	1 700	72	167	95
Concrete of ground cinders, 1 3/16 in thick	1 700	73	363	290
Cinder concrete, on metal lath, 2 in thick	1 700	66	248	182
Metal lath and patent plaster, about ½ in thick over 1 in air-space	1 700	76	296	218

ʏ York City made a series of tests on the HEAT-CONDUCTIVITY of these als. A cast-iron plate covered with the material under test was subjected mperature of 1 700° F. for two hours over a crucible furnace, and the the plate noted at regular intervals of time. The results of the tests are in Table IV on page 822.

T.C.

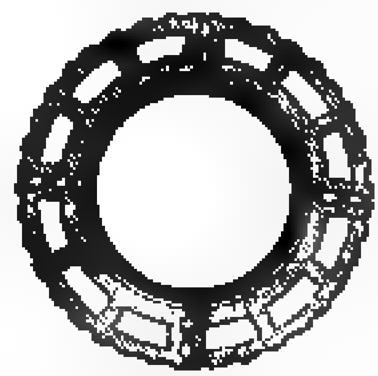
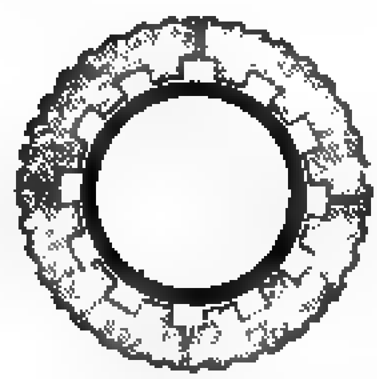


Fig. 2. Hollow-tile Protection. Cylindrical Column

lortar



set in cement mortar, occasionally, ing, bound with copper wire at inter-

. Hollow-tile Protection. Plate-and-angle Column

Fig. 3. Ribbed-tile Protection. Cylindrical Column

a-Cotta Column-Protection. Fig. 1 shows the manner in which built-mns are protected in the best class of fire-proof buildings when tile fire-; is used. Figs. 2, 3, and 4 show common methods of protecting cylin-olumns, and Figs. 5 and 6 columns of rectangular cross-section. The ard, shown in Fig. 1, is often employed in mercantile and manufacturing gs and put on to a height of 4 or 5 ft above the floor. The efficiency of struction is greatly increased by wrapping the columns with wire lath plastering, although it is not a common practice. To insure the protec-the metal under the most trying conditions, it is imperative that the

Solid-tile Pro-n. Cylindrical mn

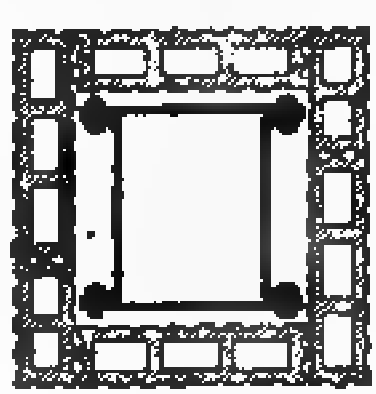


Fig. 5. Hollow-tile Protection. Built-up Box Column

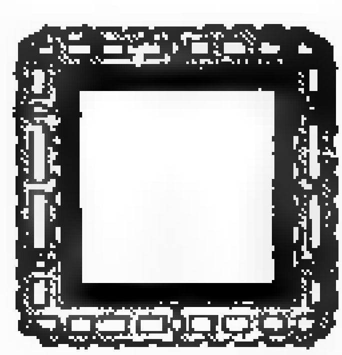


Fig. 6. Hollow-tile Protection. Square Column-section

protective covering shall not be detached by the streams from the firem hose, and thus expose the steel. This can be positively guarded against by using two layers of tiling or concrete and wrapping the inner layer metal lathing. Fig. 7 shows a column protected in this way, the construc being essentially that adopted in the Fair Building in Chicago, Ill. inner layer of tiles is wrapped with wire lath embedded in the mortar, and spaces between the and metal are f solid with cement m tar.

Fig. 7. Double-tile and Metal-lath Column-protection

Concrete Colu Protection. W concrete is to be for column-protec the way to obtain most efficient const tion is undoubted surround the metal cinder concrete, po inside of a plank f set around the colu a coat of liquid ce being first applied a brush to the m The plank form sh be set at least 2 in side of the metal. It is generally conceded that this forms one of the m efficient fire-casings for columns, and, in addition, lends added stiffness to the members embedded in it. It is advisable to reinforce the concrete or anchor i

Fig. 8. Concrete Column-protection and Wooden Form

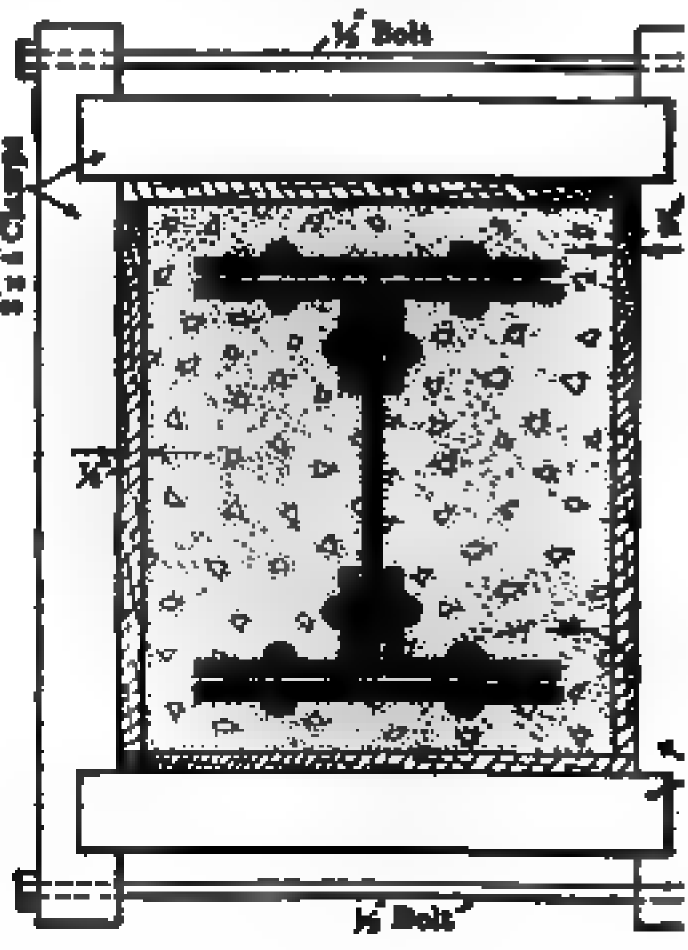


Fig. 9. Concrete Column-protection Wooden Form

ms of metal lath to the steel column. There are two general methods in in applying the concrete. Fig. 8 illustrates a column which is first wrapped ally with No. 10 gauge galvanized wire, 12 in on centers, to afford a key the concrete. The wood forms are placed the full length of the column, the concrete poured from a hole in the ceiling above. A slush-mixture of r cinder or stone concrete of 1 : 2 : 5 mix may be used. Fig. 9 shows a form ugh boards, made in sections from 4 to 6 ft in length and provided with s at each end. The concrete may be thoroughly tamped about the column ach section is placed and filled. Fig. 10 shows a method of furring the m with stiffened wire lath, which serves as a substitute for the wooden s and at the same time anchors the concrete to the steel. A similar

Furring

Concrete Column-protection. Wire-lath Furring

Fig. 11. Metal-lath and Plaster Column-protection

l may be employed to obtain an air-space by placing immediately the column an envelope of metal lath with a 2-in layer of concrete. y buildings with reinforced concrete floors, the columns are protected by plaster on metal lath. When only a single covering is provided, tection cannot properly be considered fire-proof; but when two cover- provided, as in Fig. 11, they are probably all that is necessary for cast lumns. The greatest defect in lath and plaster for fireproofing is that ter is liable to be dislodged by the force of the water from the firemen's When there are two coverings, however, this danger is reduced to a mini- (See, also, Chapter XXII, Figs. 23, 24, and 25.)

er Column-Covering. Plaster-blocks have been used in buildings as n-covering, but their use is not to be recommended While it is true ir NON-CONDUCTIVITY is in their favor, it is difficult to secure them They are easily washed away by hose-streams and subject to greater than other materials. In unimportant work their cheapness may, at stify their use.

ction of Connections between Columns and Girders. The most parts of the coverings of columns, whatever the materials used, are those about the connections with the beams and girders. Concrete edly is better adapted for covering these parts of the column than any terial, because, being elastic, it can be made to fit into any space and ny form of connection.

The Cement-Gun. During recent years, a new method of prote structural steel by means of the CEMENT-GUN has been introduced. This consists essentially of two superimposed tanks, forming two compartm from the bottom of which a dry mixture of sand and cement is ejected by pressed air through a hose-line with a nozzle at the end. To this nozzle a s hose delivers a supply of water under pressure, which is applied to the dry stituents just before they emerge from the nozzle. The mortar issuing in form of a spray shoots out from the nozzle with considerable force and pinges on the surface of the steelwork. The columns of the fifty-five-s Woolworth Building in New York City are provided with a 1½-in coatin cement mortar applied in this way, and coated on the outside with a thickness of terra-cotta. The steelwork, also, of the new Grand Central minal Buildings in New York City are protected with a 2-in coat of ce mortar or Gunnite. By this means, inaccessible corners are readily prote without the use of forms. Tests have shown that Gunnite is superior in te and compressive strength, permeability, absorption, porosity, and adhesi good hand-made products of the same kind.*

Recesses for Pipes. "As a matter of economy, both in original cost in the matter of space, it has been the common practice to run water-p waste-pipes, and vent-pipes immediately alongside the steel columns and in the fire-resisting covering."† This is undoubtedly bad construction, as Fre

FIG. 12. Tile Column-protection with Pipe-space

FIG. 13. Concrete Column-p tion with Pipe-space

illustrates by explaining its disastrous results in recent conflagrations; a the better types of fire-proof buildings, the pipe-space is now separated fro columns by the fireproofing. Fig. 12 shows a method of running the in some fire-proof buildings, and it is probably as satisfactory as any arr ment in which the pipes are to be run beside the columns. Fig. 13 sho somewhat similar method in which concrete, metal lath, and plaster are ployed for the fireproofing.

4. Fire-proof Floor-Construction

Fire-proof Floors. In the study of fireproofing-materials by far the est attention has been given to FLOOR-CONSTRUCTION; and of the very number of types which have been developed, the characteristic and le ones are here considered.

* Engineering News, 1912, Vol. 67, page 26; and Vol. 68, page 1086.
† Fire Prevention and Fire Protection. J. K. Freitag, page 374.

lequirements for a Fire-proof Floor. It goes without saying that a fire-
f floor must be made of incombustible materials. It seems unnecessary,
, to mention that it must resist as much as possible the transmission of
; so as to afford thorough protection to the metal incased by it or forming
ssential part of it. The materials used should not disintegrate or otherwise
when exposed to heat or flame. They should also resist the action of water
may be used to extinguish a fire. The floor-construction should be essen-
y water-tight, so as to prevent damage by water in stories below. It should
esigned to safely carry its load at all times. The New York City Building
: describes certain acceptable forms of fire-proof floors, but also provides for
cceptance of other forms which successfully meet the prescribed fire and
gth tests. Fully eighty tests have been made under the auspices of the New
City authorities and these, together with a few made by the authorities of
cities, comprise practically all that have been made in this country. The
h Fire-Prevention Committee of London has also made a number of such
s*

e Tests for Floors. The STANDARD FIRE TEST of the American Society
esting Materials† is essentially the same as that required by the New
City Building Code and as the one used by the British Fire Prevention
nittee. Briefly, the New York test consists in subjecting the floor in ques-
carrying a load of 150 lb per sq ft, to a fire maintained at 1700° F. for four
; and then in applying a stream of water, at 60-lb nozzle-pressure, for ten
es, the floor being considered satisfactory if there has been no appreciable
oration due to the test and if it has resisted the passage of flames during
st.

es of Floor-Constructions. In considering the several systems of floor-
uction, they are for convenience divided into the following types or
:

(1) Brick arches,
(2) Terra-cotta or tile floors:
- *a.* Segmental,
- *b.* Flat side-construction,
- *c.* Flat end-construction,
- *d.* Reinforced-tile arches,
- *e.* Guastavino,

(3) Concrete floors:
- *a.* Segmental,
- *b.* Flat reinforced floors,
- *c.* Sectional systems,

4) Gypsum floors,
5) Metal-lumber-construction.

Floor-Arches. The first attempt at fire-proof floor-construction
wrought-iron beams was made by using BRICK ARCHES sprung between
ns and resting on the bottom flanges, as illustrated by Fig. 14. When
n of construction is used the bricks should be hard, well-burned bricks,
r bricks of good shape, laid to a line on centers without mortar, with
er edges touching; and all the joints should be filled in with cement
The bricks of one line should break joints with those of the next adjoining,
ase there is more than one row, the joints of one row should also break

ist of these tests made in the United States and in London, see Proc. Am.
est. Mats., Vol. VI, page 128.
ear Book, Am. Soc. Test. Mats.

joints with those of the row above or below. The arches need not be over 4 thick for spans between 6 and 8 ft, provided the haunches are filled with a g cement and gravel concrete, put in rather wet. The RISE of the arch sho be about one-eighth the span, or 1½ in to the foot; and the most desirable s

FIG. 14. Brick Floor-arch

is between 4 and 6 ft. The building laws of many cities provide that when spans exceed 5 ft the arches must be increased in thickness, generally to 8 The HAUNCHES should be filled with concrete, level with the top of the a In first-class fire-proof construction the bottom flanges of the beams should protected by terra-cotta SKEWBACKS, as in Fig. 15 which shows the construc

FIG. 15. Brick Floor-arch. Government Printing Office, Washington, D. C.

used for the floors of the principal stories of the Government Printing Offic Washington, D. C.* A 4-in brick arch of 6-ft span, well grouted and levele with Portland-cement concrete, should safely carry 300 or 400 lb to the sq foot. Experiments have shown that brick arches will stand very severe po ing and a great amount of DEFLECTION without failure. The WEIGHT of a f such as is shown in Fig. 14, is about 40 lb per sq ft, without the concrete f finish. TIE-RODS, as described on page 865, should always be provided. brick arch is the strongest type of arch for the span it occupies, with the ex tion, perhaps, of the stone-concrete arch. It is perhaps, also, the most expen Its weight necessitates a heavier framework than is required for other t and, on account of its appearance, it is adapted only to buildings of the wareh type.

Terra-Cotta or Tile Floor-Arches. TERRA-COTTA OR TILE as a fire- material, and the relative merit of dense, porous, and semiporous tile have discussed on page 815. For floor-construction the semiporous tile is prob the best as it is a compromise between the advantages and disadvantages o dense and porous tile, particularly as to strength and fire-resistance. As

* A description of the structural features of this building may be found in the Eng ing Record for Dec. 6, 1902.

d on page 827, five different types of terra-cotta floor-construction, including ger number of systems, will be discussed. For these a great variety of es and sizes of blocks, of the dense, porous, and semiporous material, are afactured in this country. The largest company devoted to the manu-re and erection of hollow-tile fireproofing-material is the National Fire fing Company, New York and Chicago. Another large company is Henry er & Son, New York. Any one of the large companies can make any of blocks desired, except such as are covered by letters-patent, and, as a they can make them in dense, porous, and semiporous material.

lvantages of Tile Floor-Arches. Many architects prefer the use of A-COTTA ARCHES in buildings because the setting of them causes less dis-ance to the mechanics of other branches of the construction. During the ng of CONCRETE ARCHES the continual dripping of water and bits of con-interferes seriously with other work. The work of installing tile arches erally more rapid than for other types and it is not necessary to wait for to dry out. The quality of terra-cotta can be readily judged from its rance, not only before it is put in place but also after it is set. Thus it does equire the constant supervision necessary for materials that are mixed y are put in place.

advantages of Tile Floor-Arches. The principal DISADVANTAGE OF ARCHES for floor-construction is the difficulty of adapting any system to ling of irregular-shaped spaces. The arches must be set between I beams nnels, and to get the best effect the supporting beams must be parallel or so. Tile arches, especially of the END-CONSTRUCTIONS, are weakened by holes for pipes than are the monolithic floors. As there is no bond en the rows of tiles in the END-CONSTRUCTION arch, if a single tile in a row out or omitted, there is nothing to hold up the remaining tiles in the row the adhesion of the mortar in the side joints. In this respect SIDE-D arches have an advantage over the END-CONSTRUCTION. Where it is ary to use considerable concrete filling over the arch the weight of the onstruction will usually greatly exceed that of the concrete systems, and lditional weight means, also, additional expense. The floor-blocks are o breakage and chipped blocks in the floor are not unusual.

ection of Floor-Arches. Flat arches of hollow tile require close INSPEC-uring erection to see that broken or imperfect tiles are not used; that s in END-CONSTRUCTION tiles abut opposite each other; that all joints perly mortared and that all of the steelwork is properly protected. Much orkmanship has been allowed to pass in order to avoid delay, and also it cannot be discovered until the centering is removed. A tile arch ly looks better on the top surface than it does on the bottom.*

ing of Tile Floor-Arches. Tile arches are always SET on wooden s suspended by bolts hooked over the tops of the I beams. For all spans and over, the centers should be slightly CAMBERED. Before any floor-are set, all girders projecting below floor-beams should be completely on the bottom and sides, independently of the floor-construction. To the steel from rust it should have a good coat of Portland-cement mortar he tiles are applied. After the centers are in place the beam-tiles should d under the bottom of the beams and mortar slushed on the sides. The ides of the SKEWBACKS which rest against the floor-beams should then ed with just enough mortar to give them a perfect bearing, and shoved

* areless workmanship possible in the setting of tile arches was clearly set forth cle in Engineering News, April 14, 1898.

up against the beams. After this, the INTERMEDIATE BLOCKS, with their r on one end and one side covered with a full bed of mortar, should be shov into place. The KEYS* should have mortar on both sides and one end, if sn METHOD KEYS are used, and they should fit snugly, but not tight. "Under no c ditions should a key be rammed in place. It is better to use a smaller key and out the space left with either a solid slab of tile, or, if the opening is too sm with a piece of slate."* "In setting tile arches it is very common to build arches in STRING-COURSES, first fitting all the skews, then all the intermediat and finally all the keys. This is bad practice, as it loads the center, both pla and stringers, to excess, causing too great a deflection. In the END-CONSTR TION the arches should be built one by one, each being complete before next is started. In SIDE-CONSTRUCTION, where joints are broken longitu nally, the arches should be keyed up or completed at the first point where intermediates meet the lines of the key, thus completing the successive arc as rapidly as possible."† All JOINTS in the arches should be filled with mort especially at the top.

Wetting the Floor-Tiles. In warm weather all hollow tiles, whether de or porous, should be well wet or water-soaked before laying. In freezing weat they must be kept dry.

Mortar for Setting Floor-Tiles. "Mortar for setting porous hollow should never be made of cement and sand alone, as such mortar is too SHO rolls off the tile, and does not insure a full joint."* A good mortar is m by mixing the cement and sand in the proportion of 1 : 3, and adding c lime putty or hydrated lime to the extent of 10% of the cement-content. T mortar should be thoroughly worked. Hot lime mortar should never be us In dry weather the centers can be removed in 36 hours after the tiles are in pl but it is much better to allow 48 hours and even longer in cold or wet weather

Filling above Tile Floor-Arches. The strength of all tile arches is grea increased by wetting their top surface and covering it with a rich cinder c crete, mixed with Portland cement, well tamped and brought level with tops of the steel beams. If the floors are to be finished in wood, NAILING-STR are required to secure the flooring. These nailing-strips are usually dove shape in cross-section, about 2½ in wide at the top, 3½ in at the bottom from 1¾ to 2 in thick. It is preferable to lay them at right-angles to the s beams, so that they may be secured to the top flanges by metal clips, as in Fig.

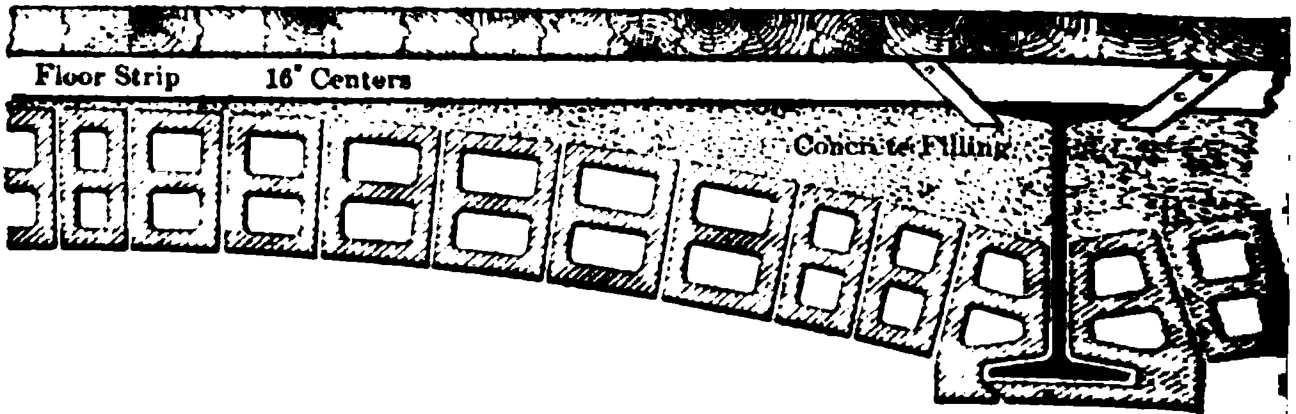


Fig. 16. Segmental Tile Floor-arch

Before the nailing-strips are laid, all piping and wiring which must go or through the tile arches should be put in place. After the nailing-stri in place the tops of the steel beams should be covered with a thin coat of

* E. A. Hoeppner. † Freitag.

d-cement-and-sand grout, applied with a brush. The spaces between the ing-strips should be filled with a 1 : 8 or 1 : 10 cinder concrete, finished ut ½ in below the tops of the strips. Some architects claim better results i strips of rectangular section, with nails driven horizontally into the ver-l sides to form the grip in the concrete. This method avoids the loosening e strips and flooring from any shrinkage of the strips.

le Filling-Blocks. In cases where the tops of the tile arches are 2 in or e below the tops of the steel beams, hollow tile blocks are sometimes used illing to the top of the beams, as in Fig. 23. These blocks are lighter than concrete, but they do not strengthen the arches.

ment Floors. If the floors are to be finished with cement, the cement and rete should be at least 2½ in and preferably 3 in thick above the steel beams, hould be blocked out in sections of not over 6 ft square, with joints extend-hrough the concrete. When practicable the joints in one direction should er the beams.

ather-Protection. Terra-cotta arches should always be protected against r snow, especially in freezing weather, as both the blocks and the mortar e joints are injured by freezing. Porous terra-cotta, especially, may be y ruined by freezing when soaked with water.

tection of Ceilings from Stains. "If plastered ceilings are to be used, rra-cotta work should be protected against the smoke or soot from the g-engines. Stains are also quite liable to occur from the effects of iron clay, or from the cinders in the concrete over the arches, if the floor is d to become wet."* To prevent these stains several kinds of hydraulic have been used, some of which have proved very effective.

mental Tile Floor-Arches. "This form of arch is the strongest and st. It is particularly adapted to warehouses, lofts, factories, sidewalks, ever great strength is required and a flat ceiling is not necessary. When strong arch is required in deep beams and a flat ceiling is also demanded, sult can be obtained by using a metal-lath ceiling suspended below the ."† These arches are usually formed by either 6 or 8-in hollow tiles, the SIDE-CONSTRUCTION principle and bonded endwise like a brick vault.

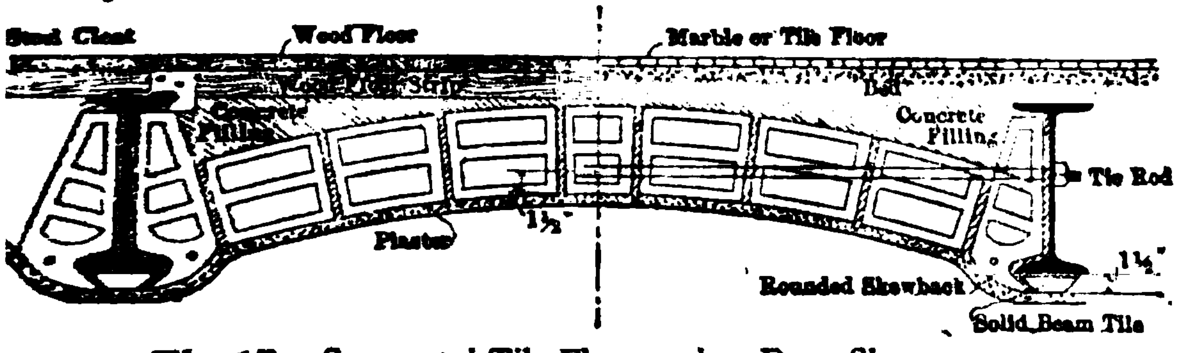


Fig. 17. Segmental Tile Floor-arch. Deep Skew

n be used for spans up to 20 ft, but it is better to limit the span to ; ft. "END-CONSTRUCTION blocks may be used, but they are unsatisfac-less the arches are of uniform span and rise throughout. The rise of -CONSTRUCTION arch can be varied by increasing the thickness of the lower part of the mortar joint, but this cannot be done with the END-CTION method."†

* Freitag.
† Bevier, National Fire Proofing Company, New York City.

Figs. 17 and 18 show typical forms of SEGMENTAL ARCHES. The w
of the arch-tiles will run about 26 lb per sq ft for 6-in tile and 32 lb for

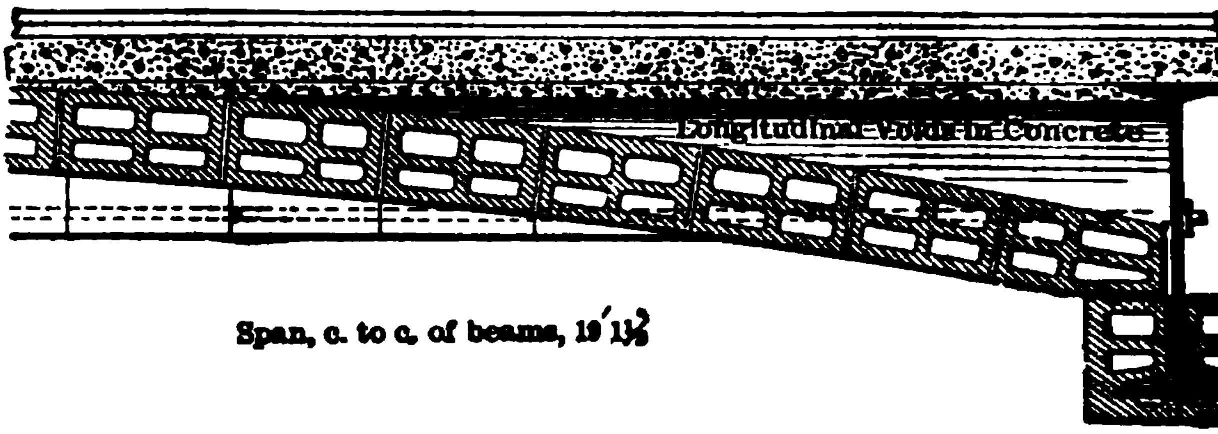


Fig. 18. Segmental Tile Floor-arch. Deep Beam. Dropped Skew

tile. To these weights should be added the weight of concrete filling, floo
plaster, etc.

Thickness of Webs. "For general use the WEBS of segment-tile shoul
½ in thick for semiporous tile and ¾ in for porous tile. The SKEWBACK sh
be at least ¾ in thick for the first-named material and 1 in for the second.
printing-establishments or any other building where a large amount of v
tion occurs the webs of all tiles must be designed in proportionate thickn
the load they are required to carry."* These thicknesses apply to Ch
practice more particularly, where a stronger tile is produced than in the East,
New York City webs are generally ⅝ in thick for semiporous and 1 in for p
tiles.

Rise of Segmental Floor-Arches. The RISE of the soffit of the arch a
the springing-line should be from one tenth to one eighth the span. The gr
the rise the less will be the THRUST of the arch. No single-cell tiles s
ever be used in any form of terra-cotta arch-construction.

Filling the Haunches. The HAUNCHES of SEGMENTAL ARCHES shou
filled with good cement concrete-leveled up to a point not less than 1 in
the CROWN of the arch. For short spans cinder-concrete filling may be
but for wide spans it is better to use gravel concrete, as the concrete
contributes to the strength of the arch at the haunches.

Tie-Rods. The THRUST of segmental arches is very considerable, so t
is important to provide TIE-RODS between the beams. A formula for dete
ing the STRESS in the tie-rods and their diameter is given on page 865.
most effective the tie-rods should be placed at the center of the skew. P
the tie-rods in this manner, however, may cause them to project belo
SOFFIT of the arch, giving an unsightly appearance to the ceiling. It i
more difficult to protect them when in this position.

Strength of the Segmental Semiporous-Tile Floor-Arches. Th
LOADS per square foot on 6 and 8-in segmental arches, with side-constr
semiporous tile, a rise of one-eighth the span, webs and shells ⅝ in thic
with a factor of safety of 7, as obtained from the tables of the Nation
Proofing Company are given in Table V.

Side-Construction Tile Floor-Arches. By this term is understo
flat-tile arches in which the voids in the blocks run parallel with the bea
shown in Fig. 19. One advantage of this arch over the end-construc

* E. A. Hoeppner.

Table V. Safe Loads for Segmental Semiporous-Tile Floor-Arches

Span, ft	6-inch arch, lb	8-inch arch, lb	Span, ft	6-inch arch, lb	8-inch arch, lb
4	1 103	1 318	11	402	480
5	878	1 049	12	370	442
6	735	883	13	340	407
7	630	735	14	317	379
8	554	662	15	296	353
9	490	585	16	278	331
10	443	529	....	....	....

e loads include the weight of construction; so that to get the safe live load, all
d load of arch-blocks, concrete fill, plastering, flooring, etc., must be deducted.

EAKING OF JOINTS that is effected in the setting of the blocks, by means ch the failure of a single block does not impair the strength of the arch l that block. The WEBS should not be less than 5/8 in thick. "RADIAL are sometimes specified but should be avoided, as they incur needless e in manufacture and endless confusion and delay in setting, without any

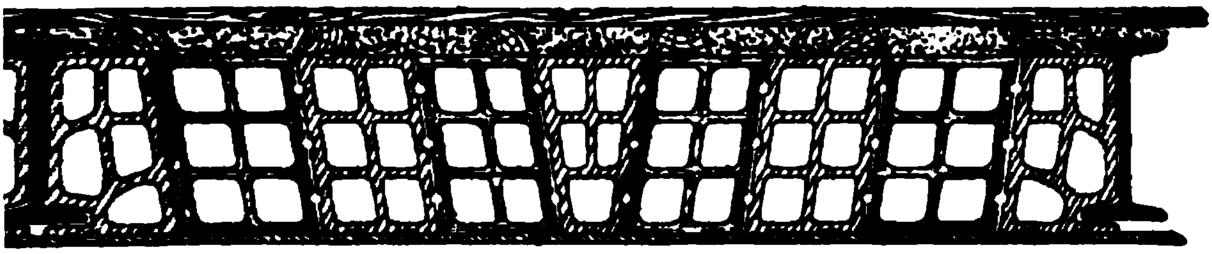


Fig. 19. Flat Tile Floor-arch. Side-construction

sating advantage." * In the SKEWBACKS a web should always be pro- cross the block at the lower flange of the beam, as at this point comes atest pressure in this block. Arches have collapsed because of failure ide this web. The DEPTH of the arch must be proportioned to the span the beams and to the load to be carried. For ordinary loads, a safe o make the depth of the block 1¼ in for each foot of span, plus the necessary for protection below the beams. SAFE LOADS for semiporous- es, side-construction, with webs 5/8 in thick and a factor of safety of 7, by the National Fire Proofing Company, are shown in Table VI.

Construction Flat Floor-Arches. In this construction the sides and the individual blocks run at right-angles to the beams, so that the pres- he blocks is endwise of the tile. It has been conclusively demonstrated low tiles are much stronger in END-COMPRESSION than transversely. jection urged against this construction is that it is wasteful of mortar cult to get the edges of the blocks properly bedded. They do require nore mortar, but the second objection is not serious, for, if the blocks o a proper bevel, the tighter they are set the stronger the arch."* The l blocks in the END-CONSTRUCTION are commonly made rectangular advancing by 1 in from 6 to 15 in in depth. The length and width, he blocks may be varied, but the standard size is 12 in for both dimen- he number of partitions or webs in the blocks varies with the size of

* Bevier, National Fire Proofing Company, New York City.

Table VI. Safe Loads for Semiporous, Side-Construction, Tile Floor-Arc

Depth of arch	6 in	7 in	8 in	9 in	10 in	12 i
Weight of arch per sq ft	24 lb	26 lb	27 lb	29 lb	34 lb	37 lb
Span of arch, ft in	Strength of arch in pounds per square foot					
4 0	197	230	263	296	438	52
4 6	156	182	208	233	346	41
5 0	....	148	168	189	281	33
5 6	....	....	139	156	232	27
6 0	....	....	....	131	195	23
6 6	....	....	....	....	166	19
7 0	....	....	....	....	....	17

These loads represent the GROSS LOADS; so that for the SAFE LIVE LOADS the w of the construction, including the arch-blocks, fill, flooring, plastering, etc., mu deducted. For blocks with thicker webs the loads may be increased proportiona Where no loads are given in the table, the spans are considered excessive fo depth of block specified. The weights of arch given in the table are for the lig blocks. If thicker webs are used, the weight of block must be taken proportion greater.

the blocks and also with the strength desired. The 6-in, 7-in, and 8-in b usually have two vertical partitions and one horizontal partition, or one ve and one horizontal, for blocks 8 in wide. The 10-in and 12-in arches may either one or two horizontal partitions. Arch-blocks over 12 in deep s always have at least two horizontal partitions. In the strongest block voids are about 3 in square. "The arch-blocks must be set end to end in st courses from beam to beam, and cannot be set breaking joints, as in the CONSTRUCTION method."*

Thickness of Web. This should be at least ¾ in for porous and ½ i semiporous tiling. The thicker the webs the greater will be the strengt fire-resistance of the arch. The end-joints are always beveled, as in Fi the ends being parallel; thus all the intermediate blocks are made with the die.

Form of Skewback. An end-construction arch may have SKEWI formed of the same blocks, with notches in the ends of the blocks to fit ov

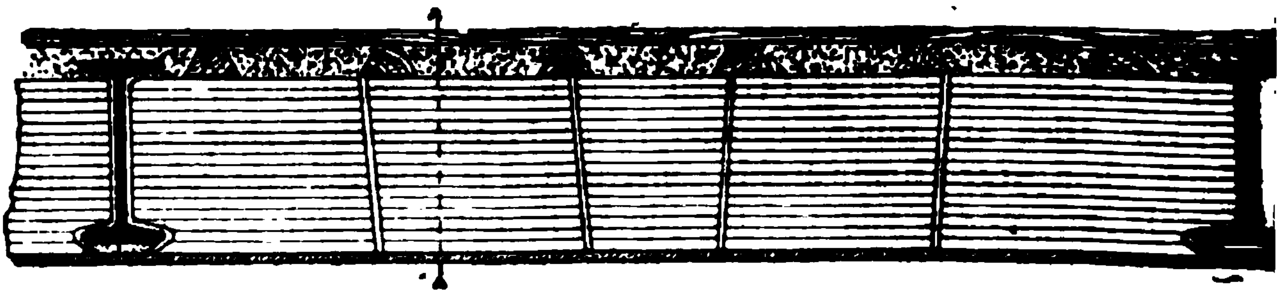


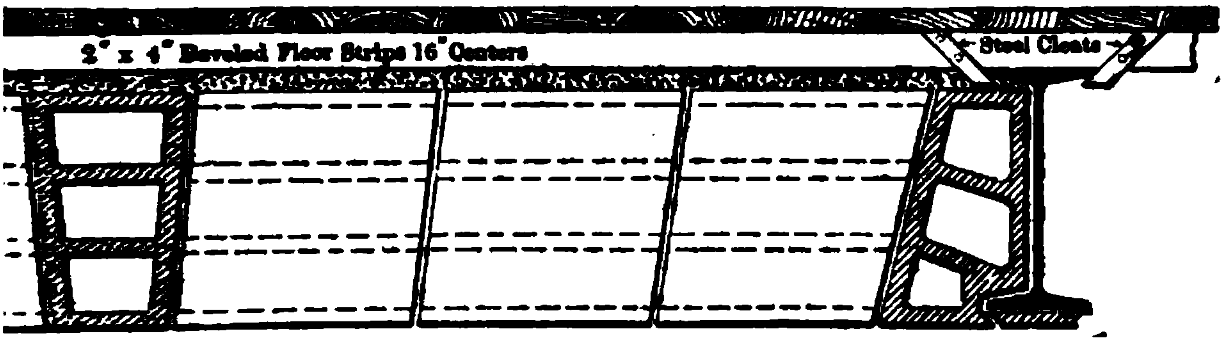
FIG. 20. Flat Tile Floor-arch. End-construction

bottom flanges of the beams, as in Fig. 20. It is generally considered th end-construction skewback is much stronger than the side-construction

* Bevier.

but on account of the large amount of mortar lost in the voids and the
ulty of obtaining an even bearing with end-construction skewbacks,
also, because of the greater facility with which the side-construction skew-
s can be used, contractors generally prefer to use the latter; and this has
rise to the form of arch shown in Fig. 21. But a more important reason for
side-construction skewbacks with end-construction arches is the better
ction against fire that they afford to the beam or girder. To develop the
sary strength, side-construction skewbacks should have a large sectional
and a sufficient number of partitions, following, approximately, the lines of
t. With any form of skewback the recess for the beam-flange should be of
width, so that when the tiles are set the protecting flanges on the skew-
will not touch the bottom of the beams, but will be at least ¼ in below
. Many varieties of side-construction skewbacks are made to meet all
le conditions.

ys. Both end-construction and side-construction KEYS are used with
onstruction arches, the choice of the key depending principally upon its



21. Flat Tile Floor-arch. Combination End-construction and Side-construction

L. If the span of the arch is such that the standard intermediate blocks
e a key 6 in or more in width, the END-METHOD KEY is used, as in Fig. 20;
the space for the key is small, a SIDE-METHOD KEY, such as shown in Fig.
used. As the key is almost entirely in compression, a side-construction key
r less in width will usually give all the strength required, provided that the
ontal webs are in the same line with those in the intermediate blocks. E. V.
on, western manager of the National Fire Proofing Company, says: "We
the use of an end-construction key in all cases where possible. Our cus-
s to use side-construction keys for spaces of 6 in and under, and end-
uction keys for larger spaces. When using the latter keys we insert a
ire-clay slab between the ends of the tile."

sed Skewbacks. Where flat arches are sprung between 18-in, 20-in, or
beams it is necessary either to use a RAISED SKEWBACK or else to have a large
above the top of the tile arches which must be filled in some way. Raised
acks are preferable to a hollow space above the tiles and cheaper than
te filling. They are often used for roof-arches, because for that pur-
is seldom necessary to make the arches as deep as the beams, while the
ust be about on a level with the beams. Raised skewbacks are almost
made on the side-construction principle. Fig. 22 shows a typical form
ed skewback for end-construction arches.

t Versus Paneled Ceilings. In connection with the raising of the
above the bottom of the beams or girders, J. K. Freitag calls attention
advantages of FLAT CEILINGS, as follows: "Flat, unbroken ceilings are
to be preferred to any type of terra-cotta arch which may require a
d effect due to the projection of the girders or beams below the main

ceiling-line." A perfectly flat ceiling reflects more light, makes a better-ligh room, and deflects the heat. Paneling forms pockets for the retention of l and flame and greatly increases the exposed area.

Fig. 22. Raised Skews for End-construction Arches

Floor-Arches and Beams of the Same Depth. A deep block makes a m stronger floor than a shallower one, and for the same depth of beams a lig and cheaper floor A 12-in arch weighs less per square foot than a 10-in with 2 in of concrete filling; and it costs less.

Depth, Span, and Weight. The MAXIMUM SPANS for different depths the AVERAGE WEIGHTS per square foot of this type of arch, set in place, ar follows:

Table VII. Maximum Spans for Flat Tile Floor-Arches of Different De and Weights

Depth of arch, in	Maximum span, ft in	Weight per sq ft, lb
6	4 6	29
8	5 6	31
9	6 0	32
10	6 6	33
12	8 0	39
15	9 0	46
16	10 0	50

The weights per square foot, as given by different manufacturers vary grea due, no doubt, to the character of the material used and to the thicknes the webs.

The DEPTH OF ARCH most frequently used is 10 in, the girders being sp to use 10-in I beams for joists spaced from 5 to 6 ft apart. As a rule the d of the arch should be about equal to the depth of the beam, as it is just abou cheap and much better construction to use deeper tiles and less concrete fillin

Safe Loads for End-Construction Tile Floor-Arches. The STREN of flat arches of hollow tile depends upon the CRUSHING RESISTANCE of the m rial, the sectional area per linear foot of arch, the depth, and the span. For t reasons it is impossible to give a table for strength which applies to all arc The values given in Table VIII for END-CONSTRUCTION arches are based u arch-blocks of the cross-sectional areas, per foot, given in the second horizo

: of the table, and are intended to have a FACTOR OF SAFETY of 7, with the ght of the tile only, deducted. Mr. Hinton says: "The SAFE LOADS as they nd in the table afford a safe general statement of SAFE LOADS FOR ALL SECTIONS e they represent specifically a light section in the case of each arch."

Table VIII. Safe Loads for End-Construction Tile Floor-Arches *

miporous material of sectional area per linear foot, as given in the second line

The loads are in pounds per square foot of floor

pth of arch in inches	6	7	8	9	10	12	15
Areas, sq in	310	340	370	400	430	490	580
Spans, ft in	lb	lb	lb	lb	lb	lb	lb
4 6	196	254	319	391	470	648	968
5 0	155	202	254	312	376	519	777
5 6	......	163	206	254	306	424	636
6 0	......	......	170	209	253	352	529
6 6	......	......	141	175	212	295	446
7 0	......	......	......	147	179	251	380
7 6	......	......	......	......	153	215	326
8 0	......	......	......	......	......	185	282

* This table is condensed from two tables prepared by H. L. Hinton.

tented End-Construction Tile Floor-Arches. Figs. 23 and 24 show ariations of a type of arch invented and patented by E. V. Johnson when

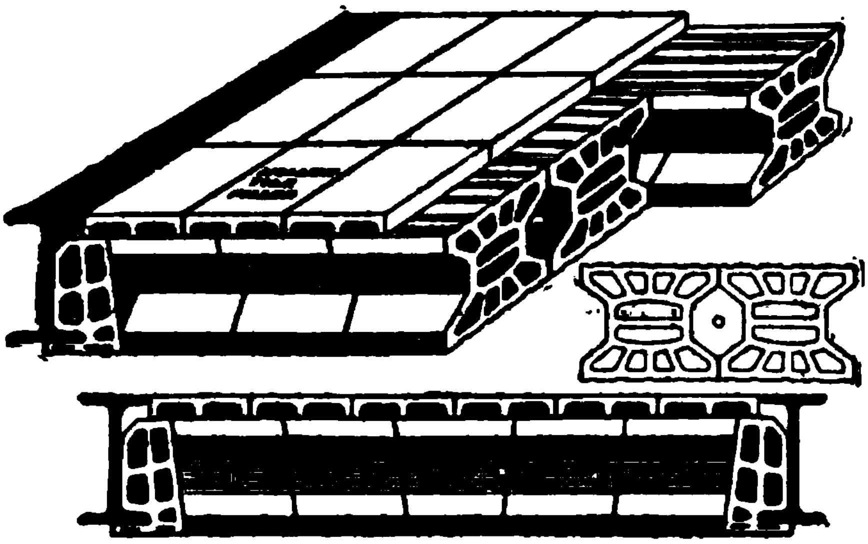


Fig. 23. Excelsior End-construction Tile Floor-arch. Side-skew

per of the Pioneer Company, Chicago, Ill. The right to manufacture and is arch, in certain territory, has been granted to the National Fire Proofing any, and to Henry Maurer & Son, New York City. The original shape of the ile is illustrated in Fig. 24. Henry Maurer & Son have modified the shape t shown in Fig. 23, as they consider that this shape gives a stronger and y heavier arch than one of the original shape. The advantages of this re the reduction in weight for an equal strength, and the clear space of 5 ween the tiles, which avoids the cutting of the blocks for the tie-rods. rch can be adapted to any span up to 10 ft by using blocks of suitable depth.

Fig. 24. Johnson End-construction Tile Floor-arch. Original Form

The LIMIT OF SPAN, WEIGHT PER SQUARE FOOT, and SAFE LOAD of the Excel
arch (Fig. 23) is given by Maurer & Son as follows:

Table IX. Maximum Spans for Excelsior Tile Floor-Arches

Depth of arch, in	Limit of span, ft	Weight per sq ft, lb	Safe load per sq ft, lb
8	5 to 6	27	300
9	6 to 7	29	350
10	7 to 8	33	300
12	8 to 9	38	350

The National Fire Proofing Company has made arch-blocks as deep as 20
and as heavy as 56 lb per sq ft. This company and Henry Maurer & Son
semiporous material for the arch-blocks. It should be noticed that the a
made by the former has an END-CONSTRUCTION SKEWBACK, while the latter us
SIDE-CONSTRUCTION SKEWBACK. The National Fire Proofing Company f
merly used the side-construction skewback, but found that when arches of t
type were tested to destruction the skewbacks were almost invariably the p
which failed, hence their adoption of the end-construction skewback. He
Maurer & Son, however, have tested, without failure, Excelsior arches of 8 ft a
10-ft spans, and with skewbacks as shown by them, with loads of over 1 000
per sq ft. These arches have been extensively used in both eastern and west
cities

Reinforced-Tile Floor-Arches. In order to obtain a wide-span flat arch
to obtain a reduced depth of arch-block for the shorter spans, the manufactu
of terra-cotta have applied to their floor-construction the principle of rei
FORCEMENT WITH METAL, which is the basis of reinforced-concrete constructi
Compared with reinforced concrete, even when cinders are used for the ag
gate, the greater depth and hollow construction of these REINFORCED-T
ARCHES secure for them greater strength per square foot for the same wei
of construction. On the other hand, however, they are undoubtedly more
pensive than cinder-concrete floor construction, because of the material u
and the increased height of the building due to thicker floors.

The Herculean Arch.* These floor-arches are built of semiporous ter
cotta blocks, 12 by 12 in on top and varying from 6 to 12 in in depth, accord

* Patented and manufactured by Henry Maurer & Son, 1898 and 1900.

: span and load. In the sides of the blocks are grooves to receive 1¼ by ⅜-in T bars. The blocks are laid end to end the entire length of the with a bearing of from 4 to 6 in on the walls or girders, presenting two uous grooves, which are filled with cement mortar, and into which the s are then inserted. The T bars must, of course, extend the full length of an. The grooves in the next course are then filled with cement mortar e blocks pushed into place, thus thoroughly covering the steel with mortar. ints between the blocks are filled with cement mortar and the blocks are o break joint endwise, as in Fig. 25. This floor has been used for spans

Fig. 25. Herculean Reinforced, Tile Floor-arch

g from 19 to 23 ft. The WEIGHT per square foot given for the terra-blocks and steel T bars is 26 lb for blocks 6 in deep, 33 lb for 8-in blocks, r 10-in blocks and 51 lb for 12-in blocks. The manufacturers estimate E LOADS for this construction as follows:

For a 12-in arch with a 20-ft span, 400 lb per sq ft.
For a 10-in arch with a 16-ft span, 400 lb per sq ft.
For a 8-in arch with a 12-ft span, 150 lb per sq ft.

CHIEF ADVANTAGE of this construction is said to be its low cost as com-with the cost of systems equally fire-proof and requiring steel beams or 8 ft. It is particularly well adapted to buildings with masonry walls titions, as in such buildings little or no structural steel is required. The nstruction affords, also, an unusually smooth undersurface, thereby g the cost of plastering. No TIE-RODS are required for this floor.

Johnson Long-Span Flat Floor-Construction. This REINFORCED-OOR was invented by E. V. Johnson, and is now controlled and erected National Fire Proofing Company. Its general construction is as follows: orary flat centering is first erected, and over this is spread a layer of rtland-cement mortar about ¾ in thick. On top of this mortar is laid N FABRIC containing steel rods varying from ¼ to ½ in in diameter, ng to the span, and spaced from 2 to 8 in, center to center. Another the same mortar is then spread on top and hollow tiles, from 3 to 12 in in according to the span, are then set in the mortar and laid so as to BREAK

joint and to form continuous rows from one support to the other. A lay of concrete, also, about 2 in thick, is usually spread on top of the tiles. Fig. 2

Fig. 26. Johnson Reinforced, Tile Floor-arch

shows the general method of construction of this system, but without the rod which are inserted in place as the fabric is used. For short spans the fabr can be used without the rods. This system differs from the flat concrete syste only in the substitution of hollow tiles for the concrete in the upper portion the slabs, the strength of the floor depending upon the REINFORCEMENT ar the ADHESION of the cement mortar to the steel and tiles. As the tiles a covered both on the bottom and top with concrete, the FIREPROOFING PRO ERTY, also, is measured by the resistance of the concrete and not by that of th tiles. Tests have shown that the ADHESION of the mortar is perfect and that will stand a high temperature without injury. This construction can be us for any span up to 25 ft, the most ADVANTAGEOUS SPAN being about 16 ft. Th WEIGHT per square foot, including the fabric and the cement on the bottom ar in the joints, but not on top of the tile, is as follows:

Depth of tile, inches............	12	10	9	8	7	6	5	4
Weight per square foot, in pounds	60	55	45	42	37	34	26	24

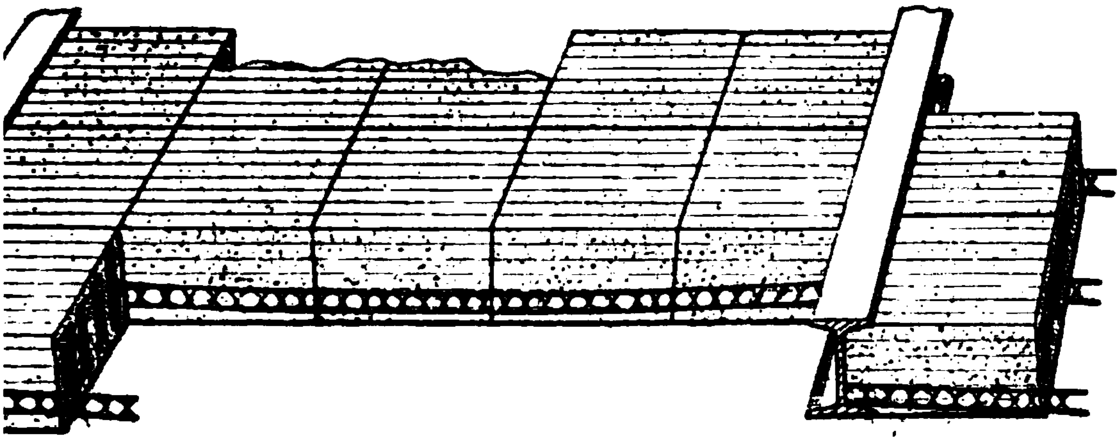
The concrete above the tile should be figured at 12 lb per sq ft for each in in thickness. The STRENGTH of the floor, with 1 in of 1 : 3 Portland-ceme mortar on top of the tiles, is given in Table X.

The New York Reinforced-Tile Floor-Arch. This arch (Fig. 27) w designed by P. H. Bevier, of the New York City branch of the National Fi Proofing Company, for use "when a light and cheap but strong floor constru tion with a flat ceiling is required, and is particularly adapted to wide spa in shallow beams. When light floor-construction with deep beams is necessa it can be secured by setting the blocks level with the tops of the beams and usi a flat metal lath ceiling, or by omitting the ceiling a panel effect is obtaine When shallow beams are used the blocks are set level and 1 in below the botto of the beams. Light cinder concrete or dry cinders are used to level up to th top of the beams. A WIRE-TRUSS REINFORCEMENT, similar to that shown in Fi 28, used in this system, is shipped to the building in reels, and is cut to prop

ble X. Ultimate Strength of the Johnson Floor-Construction

Thickness of tiles in inches								
12	10	9	8	7	6	5	4	3
Ultimate strength in pounds per square foot								
3 375	2 580	2 140	1 850	1 525	1 265	1 000	775	560
2 800	2 340	1 780	1 536	1 264	1 052	832	640	464
2 350	1 800	1 480	1 280	1 064	880	700	540	390
2 000	1 540	1 265	1 100	910	752	595	460	334
1 730	1 325	1 100	950	780	650	510	400	290
1 500	1 160	950	830	680	590	450	348	250
1 320	1 010	840	720	600	500	395	305	220
1 180	900	740	640	578	440	350	270	194
1 020	795	664	570	473	392	310	242	174
844	645	535	462	381	314	250	194	.....
700	536	445	384	316	263	208	.....	.....
587	450	370	320	266	220	.....	.....	.....

he job as required. It is embedded in Portland-cement mortar be-
locks, so that it is protected both against rust and fire. The open-



ew Plain-Skew

Fig. 27. New York Reinforced, Tile Floor-arch

iction of the WIRE TRUSS enables the mortar to flow freely all about
int can be thoroughly filled between the blocks, and the wire perfectly

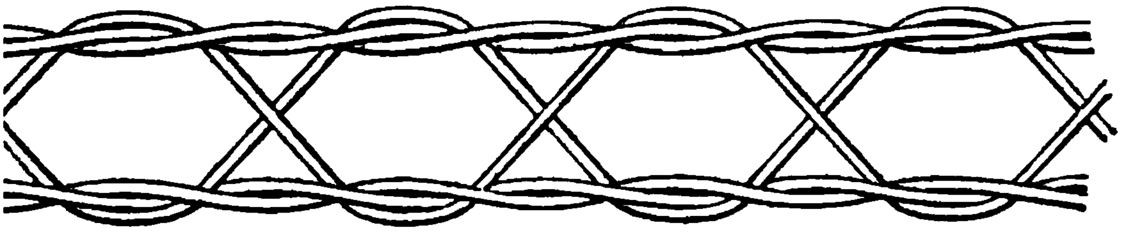


Fig. 28. Wire Reinforcement for New York Floor-arch

The floor has successfully passed the fire and load tests of the
uildings, of New York, and as a result has been used in a number of
gs in New York. Load tests were made to determine the ULTIMATE
the 6-in arch on a 6-ft span, and it was found to be 1 600 lb per sq ft."

stavino Tile-Arch System. This is a method, devised by R.
of New York and Boston, of constructing floors, partitions, stair-

cases, etc., by means of THIN TILES, 1 in thick, about 6 in wide, and from 12 t 24 in long, all bonded together in Portland-cement mortar so as to make o solid mass. The floors are built by spanning the spaces between the girde with single arches, vaults, or domes, constructed of two, three, or more thic nesses of 1-in tiles, the number of thicknesses depending upon the dimensio of the arches or vaults. In its best application, steel is used in tension only i tie-members; and in place of steel girders, tile girders are constructed of th same material. Wherever steel is used it is embedded in the masonry constru tion. One of the earliest notable buildings in which this system was used is th Boston Public Library Building, completed in 1895. Some of the later importa constructions are the Cathedral of St. John the Divine, New York City; th Minnesota State Capitol Building, St. Paul, Minn.; the Girard Trust Company Building, Philadelphia; the Chicago and Northwestern Railway termin station, Chicago; the Pennsylvania and the New York Central Railroad te minal stations, New York City; and the Hall of Fame, University of New Yor New York City.

An illustration of the wide spans that can be safely used with this system construction is seen in the Cathedral of St. John the Divine in New York City The floor above the crypt, measuring 56 by 60 ft, with no interior support and designed to carry a safe load of 400 lb per sq ft, was constructed on th principle. Wherever a VAULTED CEILING is desired this form of constructio seems to be well adapted for use. Floors built in this way have been teste under the supervision of the New York City Building Department up to 3 700 l per sq ft, on spans of 10 ft. When used between I beams the only steel bean required are those spanning from column to column. Architects contemplatin the use of this system of construction are advised to consult the R. Guastavin Company before letting any contracts. Wherever vaulted ceilings are require this construction should be at least as cheap as any other form of equally fir proof construction, and it is often cheaper. One particular advantage of th system is that frequently the soffit-course of tile is of PRESSED OR GLAZED MATE RIAL, making a most effective and permanent finish, as in the case of the Cit Hall station of the New York City subway. This station was constructed f very heavy loads and without the use of steel.

Incidentally, attention may be called to the RUMFORD TILE developed in co nection with this construction, to be used as the first course of tile, that is, o exposed surfaces on the interior of auditoriums, on account of its sound-absorbin character. Professor Sabine of Harvard University concluded from his in vestigations that this tile "has over sixfold the absorbing power of any existin masonry construction, and one third the absorbing power of the best-know felt."*

Concrete Floors. Concrete used in fire-proof floors may be either PLAIN o REINFORCED. Without reinforcement its use is generally practicable for ver short spans only, on account of its weight. In this chapter it is considered onl as a FLOOR-FILLING between steel beams. Chapter XXIV is devoted to a di cussion of the principles governing the design and use of reinforced concrete.

Advantages of Reinforced Concrete for Floor-Construction. Althoug many ADVANTAGES are claimed for reinforced concrete over the tile system the principal advantage is that of economy, taking into account the cost o both the steel framework and the filling between. The other important ad vantages are less weight per square foot of floor (usually but not always th case), adaptability to irregular framing, and rapidity of construction. Excep in the immediate locality of the tile-factories, fire-proof floors of concrete c

* The Brickbuilder, January, 1914.

ally be placed at less expense than is incurred in setting floors of hollow tile; when the spans permit the use of cinder concrete, the concrete floors are ter than those of the tile, when both floors have the same strength. Some he long-span tile-systems, on the other hand, are much lighter than many e concrete floors that are now being built. The materials entering into the struction of reinforced-concrete floors are readily obtained in almost any lity, no specially prepared material is required, except perhaps in a few ial forms of reinforcement, and the work can be done almost entirely by illed labor. Less capital is required for concrete work than for the tile-structions, and no material need be carried in stock during an idle period, pt tools, mixing-machines, old centering, etc. That the above advantages real is sufficiently proved by the immense amount of reinforced concrete under construction throughout the world. Wherever a floor is to have a hed, cement surface, reinforced-concrete constructions are considerably per than any tile system, because in the former, the entire concrete is used ive strength, while with the flat-tile arches it merely increases the dead weight.

isadvantages of Reinforced Concrete for Floor-Construction. One ded DISADVANTAGE connected with concrete floor-construction is the inter-ce in a large measure with the progress of other parts of the work. During nstallation, there is a constant dripping from the floor, making it some-s impossible to continue other lines of work. After the completion of the s a long time is required, depending upon the weather, for the drying out, e interior finishing can proceed.

omposition of the Concrete. The materials used for concrete are dis-ed on pages 240 to 241 and on page 817. Portland cement, only, should ed in any floor-construction. For most reinforced-concrete floors, having an between the steel beams of 8 ft or less, CINDER CONCRETE is generally for the reason that concrete mixed with cinders is much lighter than that d with broken stone or gravel. The usual PROPORTIONS OF CINDER CON-E are one of cement, to two of sand, and five or six of cinders. For a first-concrete the cinders must be screened through a mesh not larger than n, and only hard-coal cinders should be used. Good cinders may some-s be obtained from power-plants using soft coal, but they must be well ened and free from ash. Concrete mixed with common ashes, a mixture oc-nally used, has little strength and is totally unreliable. For all spans ex-ing 8 ft, either GRAVEL OR BROKEN ROCK should be used, and these should ixed with one part cement, to two of clean sharp sand, and four of stone avel. The WEIGHT OF CINDER CONCRETE will vary from 80 to 110 lb per t, depending upon the coarseness of the material, the quantity of sand, and amount of tamping. For ordinary purposes a 1 : 2 : 5 cinder concrete ld be used, weighing 96 lb per cu ft, or 8 lb per sq ft per inch of thickness.

rms of Reinforcement. While steel in small sections is used almost ely for the reinforcement, there is a great variety in the shape and char-r of the metal employed. Different FORMS OF REINFORCEMENT are described discussed in Chapter XXIV. All of them may be used, and most of them ow being used in floor-construction. In addition to those forms discussed , others that are not readily adapted to beam-construction are used in -construction. Such are the METAL FABRICS described farther on under ifferent types of construction. The proper position for the reinforcement floor-construction is that in which it will take the TENSIONAL STRESSES, in floor-slabs, near the lower surface. The most logical form is that of OR BAR. A greater number of small rods or bars is preferable to a smaller ber of larger ones, because the proportion of the AREA OF ADHESION between

steel and concrete to the SECTIONAL AREA OF STEEL is greater in the former
This result is apparently attained in systems in which wire fabrics are
But the disadvantage in the use of the smaller reinforcement is the greater
sibility of CORROSION and consequent failure of the construction. There
further disadvantage in the use of wire fabrics; they are easily displac
the process of placing of concrete, either getting too low and becoming ex
to fire or corrosion, or getting too high with a corresponding weakening o
floor. Another detail that must be remembered when using metal fab
that the mesh must be large enough to allow a good BOND to be formed bet
the concrete above and below it. Reinforcements in the form of bars set
tically in the concrete have a tendency to SHEAR through slabs which are
heavy loads. The best and most LOGICAL REINFORCEMENT for fire-proof
consists of from ½ to ¾-in round or square rods, either plain or defor
spaced at varying distances to suit the spans and loads.

Necessity for Cross-Bars. Where wire strands or bars are used for forcement it is essential to have CROSS-BARS as well as TRANSVERSE TEN BARS, because, when the loads are heavy and concentrated, or when a h body falls upon a slab the concrete will crack between the carrying bars. can be readily demonstrated by testing with a drop-test a floor-slab that h cross-bars. When the load is UNIFORMLY DISTRIBUTED the cross-bars ar brought into play; floor-loads, however, are more often CONCENTRATED UNIFORMLY DISTRIBUTED

Segmental Concrete Floor-Arches. For heavy warehouse-floors ARCH SYSTEMS are preferable to the FLAT SYSTEMS, because in the forme concrete is used in its strongest form, and less reinforcement is required. warehouses, also, a ceiling formed of a series of arches is not objection For spans between floor-beams of 5 ft or less, a 1 : 6 gravel-concrete arch, thick at crown and without any reinforcement, should sustain, without c ing, a distributed load of 1 500 lb per sq ft. For spans exceeding 5 ft, the brated Austrian experiments (1891–1892) seem to show that the reinforci concrete with small I beams adds greatly to the strength of the arch; but small rods or netting are not of sufficient advantage to warrant the addit expense.* Tests made on arches of 8-ft span gave the following results:

A concrete arch, 3⅛ in thick, 9½ in rise, broke at 1 130 lb per sq ft. A M arch (wire netting), 1 15/16 in thick, 10¼ in rise, or about one half the thic of the concrete arch, failed at 1 217 lb per sq ft. A brick arch, 5½ in t 9.85 in rise, failed at 885 lb per sq ft. A hollow-brick arch, 3 15/16 in thick, in rise, failed at 401 lb per sq ft. A concrete arch, 13-ft span, 3 15/16 in thick, in rise, failed at 812 lb per sq ft. A Melan arch, 3⅛ in thick, 11.4 in rise, b at 3 360 lb per sq ft. The Melan arch had I beams 3⅛ in deep, spaced apart. The structure was one year old when tested.

The concrete arch, considered as a monolithic construction, if built of concrete, is superior to the brick arch. The cinder-concrete arch is in only in point of strength. Such an arch should be at least 4 in deep a crown, and the rise should be not less than one eighth the span. Cinder crete should not be used for spans exceeding 8 ft. The strength of such a for ordinary cinder concrete is about the same as that of a 6-in segmenta arch of the same span, as given in Table V. All arch systems, whether of co or tile, require tie-rods between the beams to take up the thrust of the a (See page 865.)

Weight of Segmental Concrete Arches. The weight of solid segm

* See Architecture and Building, Jan. 4, 1896.

...s may be found by the following formula which gives results approximately ...t when the rise of the arch is not more than one sixth of the span:

$$W = (w/12)(c + 4S/p)$$

...ich

W = weight of arch, in pounds per square foot;
w = weight of material, in pounds per cubic foot;
c = thickness of arch at crown, in inches;
S = span of arch, in feet;
p = ratio of span to rise of arch.

...ble XI gives the weight per square foot of arches having a thickness of 4 in ...crown and constructed of stone or gravel concrete, taken at 144 pounds per ...foot, for various spans and ratios of span to rise. For greater thicknesses ...crown these weights should be increased by 12 lb for each inch of additional ...ness. For other materials the weights are directly proportional to the ...ts of the materials. Thus, if cinder concrete weighing 102 lb per cu ft is ... the weight of the arch for any particular span and ratio of span to rise ...2/144, or 17/24, of the weight given in the table for the same span, ratio, ...hickness at the crown. Cinder concrete of good quality weighs, according ...nsity, from 96 to 108 lb per cu ft.

Table XI. Weight per Square Foot of Segmental Concrete Arches

Concrete taken at 144 lb per cu ft

...tio of ...an to rise	Thickness of arch at crown, in	Span in feet					
		5	6	7	8	9	10
6	4	88	96	104	112	120	128
6½	4	85	93	100	107	114	122
7	4	82	89	96	102	109	116
7½	4	80	86	93	99	106	112
8	4	78	84	90	96	102	108

...t Reinforced Floors. These floors consist of slabs of concrete, varying ...hickness according to the span and load, constructed between the steel ...-beams and reinforced near the lower surface with steel in one of the shapes ...red to on page 843, and further described under their respective names. For ...nary loads the thickness of the slab should be at least ⅝ in for each foot of ..., with a minimum thickness of 3½ in. Thinner slabs have been used, but ...thickness should be carefully considered for each particular case. The floor-...s are not usually of the same depth as the beams supporting them. The ...tion of the slabs, therefore, determines the character of the ceiling. When ...bottom of the slabs is placed at or below the lower flanges, a flat ceiling results, ...the space over the slabs must be filled to the underside of the flooring with ...incombustible material, thus often increasing the weight. When the slabs ...t at the top flanges, there is a paneled ceiling, unless a hung ceiling is pro-...d.

...rength of Flat Floor-Construction. The following empirical formula, ...esenting the practise established by the New York Building Code, is based on ...nvestigation of cinder concrete floor-construction made by Harold Perrine

and George E. Streban,* under the joint auspices of Columbia University the Bureau of Buildings, Manhattan, New York.

$$w = Kda/S^2$$

in which

w = safe load, in pounds per square foot, including the weight of slab;
d = distance, in inches, from top of slab to center of reinforcement;
a = cross-sectional area, in square inches, of the reinforcement, for foot of width of slab;
S = span, in feet, of slab;
K = a coefficient with values as follows: when cinder concrete is 26 000 if the reinforcement consists of steel fabric continuous supports; 18 000 if the reinforcement consists of steel rods or shapes securely hooked over or attached to the supports; and 1 if the reinforcement is not continuous over the supports; and stone or gravel concrete is used, 30 000, 20 000, and 16 000, res ively, for the corresponding conditions.

The material contemplated by this formula is a concrete consisting of one of Portland cement, and not more than two parts of sand, and five parts of s gravel, or cinders. The reinforcement consists either of steel rods or suitable shapes, or steel fabric. In case cold-drawn steel fabric is used, th sional reinforcement should not be less than 1⅓00%, and in case other for reinforcement are used, not less than 2⅓00%, the percentage being based o sectional area of the slab above the center of the reinforcement. For p protection against fire and corrosion the center of the reinforcement should least 1 in above the bottom of the slab, but there should always be at least of concrete outside of any part of the reinforcement. The formula should n applied to spans exceeding 8 ft. Cinder-concrete floors should be limit that span in any case.

Expanded Metal. This material is now so well known that it requires a brief description. The diamond mesh shown in Fig. 29 is used in floor

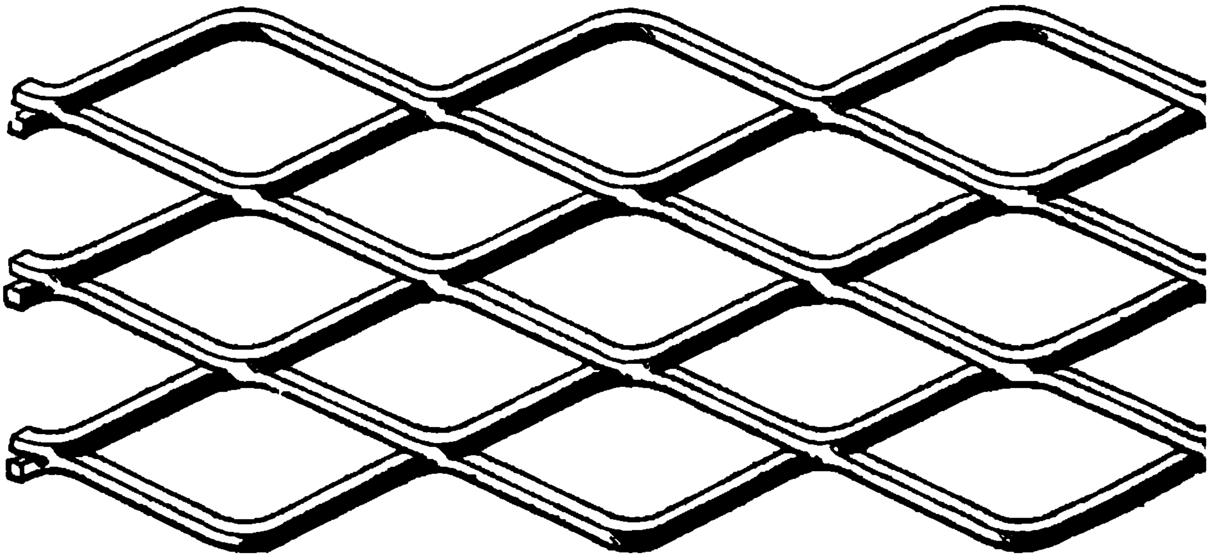


FIG. 29. Expanded Metal, Diamond Mesh

struction. For this purpose the 3-in mesh is used, the size of the mesh designated by the width of the diamond-shaped spaces. It comes in 8, 10, 12, and 16 ft long, and from 3 to 8 ft wide, according to the width mesh. It is made from a soft, tough steel of fine texture, varying in thic from No. 13 to No. 1, Stubbs gauge. The standard sizes offered by the

* Trans. Am. Soc. C. E., Vol. LXXIX. 1915, page 523.

ited Expanded Metal Companies and the Northwestern Expanded Metal mpany are in accordance with a decimal variation in cross-section, thus: 5, 0.30, 0.35, 0.40, etc., sq in per ft of width. The designations of the sizes cate the cross-sectional areas per foot of width, thus: 3–9–20 denotes a 3-in sh, No. 9-gauge plate, and a cross-sectional area of 0.20 sq in per ft of width. Sharon Steel Hoop Company and also the General Fire Proofing Company r from eight to ten sizes of expanded metal with a range sufficient to take t of the needs of concrete-floor designs.

Concrete and Expanded-Metal Floor-Construction. Of the numerous es of floor-construction possible with expanded-metal reinforcement, the shown in Fig. 30 is generally used and recommended. At the right hand

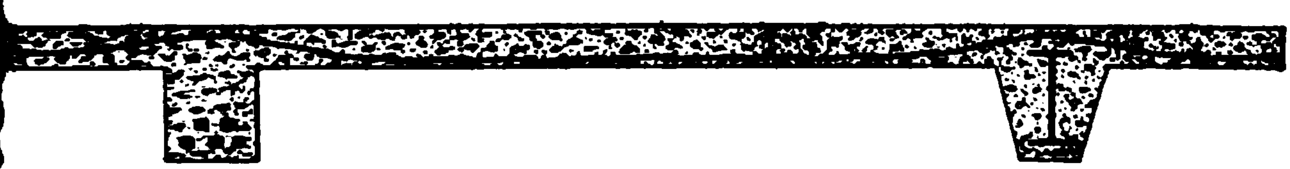


FIG. 30. Concrete Floor-construction. Expanded-metal Reinforcement

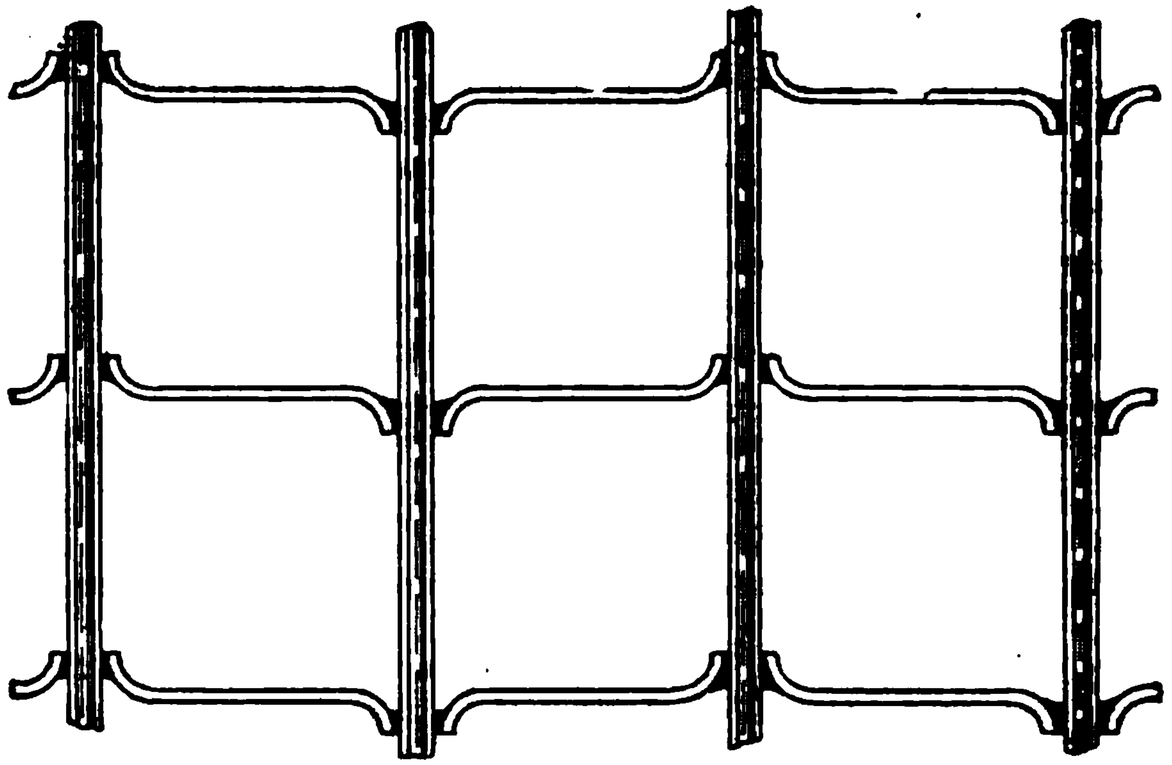
the figure is shown the construction when there are steel beams, and at the hand when there are reinforced-concrete beams. The advantages claimed expanded metal as a reinforcement are: a better arrangement in the con- e than is possible with an equal amount of material in any other form; great iency in the carrying of concentrated loads, due to the obliquity of the nds; a uniform distribution of small sections at frequent intervals, pref- ble to larger sections at greater intervals; an increased ultimate strength high elastic limit, due to the method of manufacture, thus combining advantages of a low-carbon steel with a high ultimate strength; and a hanical bond with the surrounding concrete. When used between I beams, hout other reinforcement, the spans usually vary from 6 to 8 ft, although ls 12 ft wide between beams have been constructed. In placing expanded al in the concrete, it is necessary to lap the sheets on the ends up to and uding 3–9–20, one diamond (8 in); from 3–9–25 to 3–6–60, one and a half monds (12 in); and heavier than 3–6–60, two diamonds (16 in).

Table XII. Properties of Rib-Metal

Size-number	Width of sheet, in	Area of metal per foot of width, sq in
2	16	0.540
3	24	0.360
4	32	0.270
5	40	0.216
6	48	0.180
7	56	0.154
8	64	0.135

ib-Metal. The Truscon Steel Company's factory, at Youngstown, O., is ufacturing a steel reinforcement for concrete floors consisting of a series traight ribs or main tension-members rigidly connected by light cross-ties nded from the same sheet of metal in the form of a mesh (Fig. 31). It anufactured from medium open-hearth steel in seven sizes of mesh, 2, 3, 4,

5, 6, 7, and 8 in, and in lengths up to 18 ft. It is supplied in either flat or cur sheets, and longer lengths and special sizes of mesh can be provided. The wi



Area of rib 0.09 sq in
Ribs spaced 2, 3, 4, 5, 6, 7, and 8 in
Fig. 31. Rib-metal Reinforcement for Concrete Floors

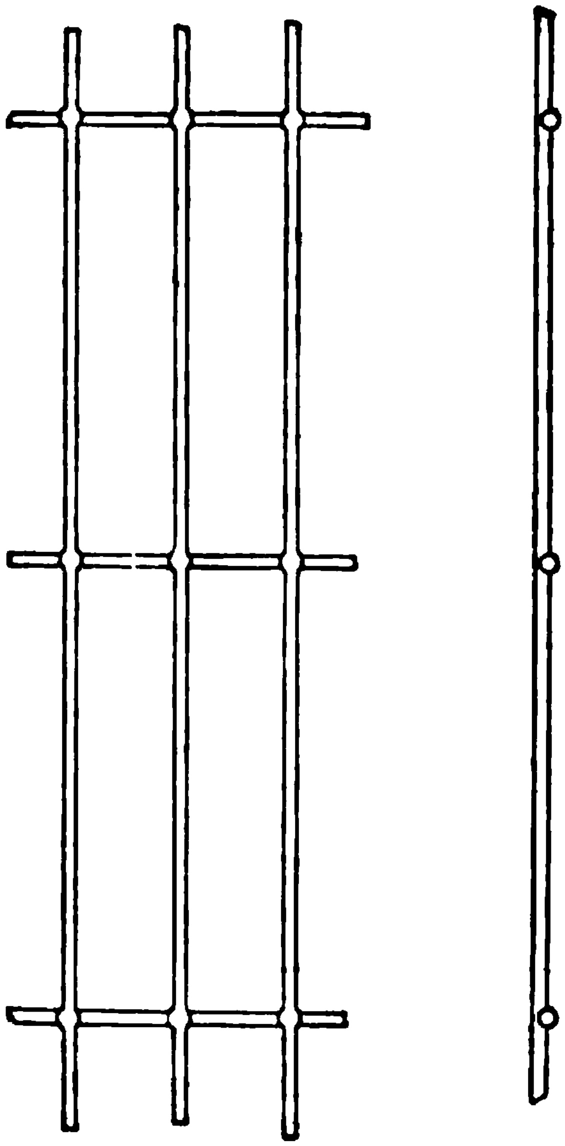


Fig. 32. Welded-metal Fabric. Clinton Wire Cloth

of sheet is governed by the size of m there being nine bars or ribs in each sh

Welded-Metal Fabric. The Wick Spencer Steel Corp. manufactures a wel fabric or mesh which has been extensiv used in the United States as a reinfo ment for concrete construction of all ki Fig. 32 shows the general style of the fal the meshes and wires of which can be va indefinitely, upwards from a 1-in m The advantage claimed for this fabric reinforcement for slab-construction is the carrying wires may be varied, bot size and spacing, to give the necessary for any given weight and span, and distributing or cross-wires, also, may varied in the same way. The directio the wires coincides with the LINE OF ST so that there is no tendency to distort rectangle of the mesh. The cross-w being welded to the carrying wires, rigidly held in place and prevent the fr from slipping in the concrete. The c is made that the elongation that takes in the carrying wire under the stress of h loading, is divided along the carrying as often as the cross-wires occur, instea being concentrated at one point as i case with loose rods or wires. In the m

monly used the carrying wires vary from No. 10 to No. 3 in size (Washburn Moen gauge), and are spaced from 1 to 4 in on centers; while the distributing wires vary from No. 11 to No. 6 in size, and are spaced from 3 to 12 in on ters. Welded metal is manufactured in long rolls, and by its use all joints laps are avoided. A floor can be made with a continuous metallic bond from to wall, that is, when the mesh is laid over the tops of the steel beams. The th of the rolls varies from 48 to 86 in.

Lock-woven Steel Fabric. This fabric* is made up in a rectangular mesh, usual spacing of the longitudinal wires being 3 in on centers and that of the nsverse wires 12 in on centers. These spacings can be easily varied to meet

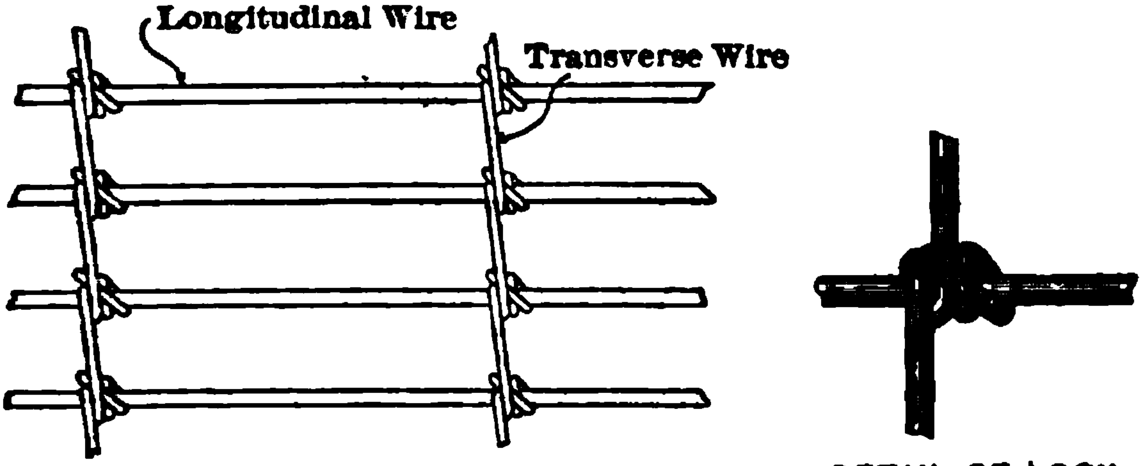


FIG. 33. Lock-woven Fabric

cial conditions. The fabric is usually made 54 in wide and comes in rolls taining from 150 to 600 lin ft, the 150-ft length being commonly used. While usual width of the fabric is 54 in, it can be varied in multiples of 1½ in from n up to a maximum of 54 in. The longitudinals or carrying wires of the ric are held in place by the transverse wires as shown in Fig. 33. The longi-

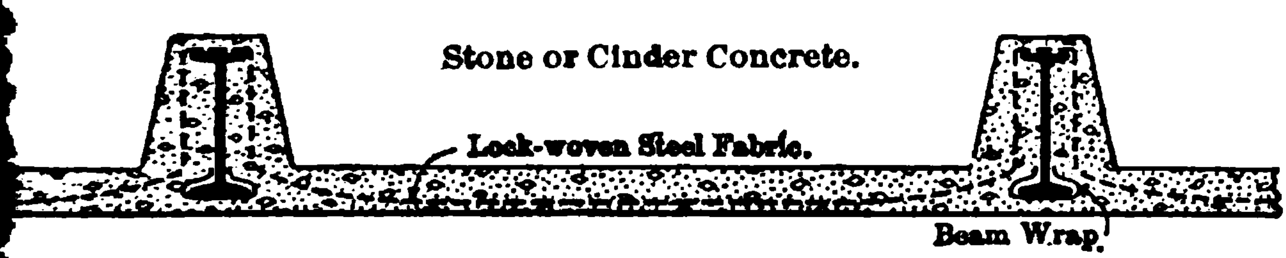


FIG. 34. Concrete Ceiling-slab Reinforced with Lock-woven Fabric

nal wires can be furnished in sizes varying from No. 14 to No. 7 gauge, the ional area of the fabric ranging from 0.0201 sq in to 0.1968 sq in per ft of th. Heavier fabric can be furnished to meet special conditions. The transe wires are usually No. 14 or No. 12. The longitudinal wires are made by a ial process which gives them an ULTIMATE TENSILE STRENGTH of from 150 000 80 000 lb per sq in, with a correspondingly high ELASTIC LIMIT. The fabric be furnished either black or galvanized. This fabric has the general advans common to any continuous, rectangular-mesh material, as it provides a TINUOUS BOND from end to end of a structure, and the wires are so placed that y lie parallel to the LINES OF STRESSES which they are called upon to carry. The dard type of construction for floor-slabs and roof-slabs is similar to that n in Fig. 30 for expanded metal. Where a flat ceiling is desired the type onstruction shown in Fig. 34 is very useful. Both of these types have been ed by the Bureau of Buildings of the City of New York on spans up to and

* Controlled by W. N. Wight & Company, New York City.

including 6 ft, for live loads running from 130 to 330 lb per sq ft; and on spa
7 ft, approvals have been given up to 175 lb per sq ft, and on spans of 8 ft, u
150 lb per sq ft. The arches were constructed of cinder concrete and the fig
given are based on a factor of safety of 10. In addition to its use for the
struction of floor-slabs and roof-slabs, the fabric is suitable for use in panel-w
sewers, penstocks, and tanks, and in all other places where a sheet-reinforcem
can be used to advantage.

Triangle-Mesh Wire-Fabric Reinforcement. Under this name the Am
can Steel and Wire Company is manufacturing a wire fabric of cold-drawn
wire for the reinforcement of fire-proof floors. A detail of the standard mat
is shown in Fig. 35. The triangular mesh is built up of either single or stra
longitudinals with the cross-wires or bond-wires running diagonally across
width of the fabric. It is claimed that the triangular mesh affords an
distribution of the steel, reinforcing in every possible direction, and that

FIG. 35. Wire-fabric Reinforcement, Triangular Mesh

strength is increased by reason of the truss-construction. For floor-reinf
ment, this fabric is used the same way that any of the other fabrics previo
described are used, and as indicated in Figs. 26, 30, and 34. The longitu
wires in Triangle Mesh are invariably spaced 4 in on centers, but the diag
wires may be spaced either 4 or 8 in apart. The manufacturers can fu
different styles, giving variations in the cross-sectional area from about 0.0
in to about 0.395 sq in per ft in width of the fabric, or a variation in weigh
square foot of from 0.2 to 1.6 lb. The material is furnished either galva
or plain. The longitudinal wires are made of either a single wire or of tw
three wires stranded. The cross-wires or bond-wires are of either No. 1
No. 12¼ gauge. Special sizes of additional area can be furnished upon app
tion to the company. This fabric is said to have an ultimate strength of no
than 85 000 lb per sq in.

Dovetailed Corrugated Sheets. Ferroinclave. Sheets of thin steel c
gated so as to form dovetailed grooves have been used as a reinforcement and
tering for concrete-steel, the dovetailing serving to unite the sheets to the
crete. The Brown Hoisting Machinery Company of Cleveland, Ohio,

tented, under the name Ferroinclave, a tapered corrugation which is small ugh to hold hard mortar, and hence can be plastered on the under side. Fig. shows a partial section of the Ferroinclave corrugated sheets, the depth of the rugations being ½ in, the distance from center to center of corrugations 2 in, d the corrugations, with the opening between the edges, ⅞ in. The tapering of e corrugations is of especial advantage for roofs, as it allows the sheets to be pped at the end-joints, making a roof absolutely tight, even if water should etrate the cement coating. The principal advantage in the use of corru- ted sheets for floor-construction is that they sustain the concrete, when the ns are of moderate width, before it has set, thus saving the cost of centering d the time required to put it in place. This advantage, however, appears to offset by the high cost of the sheets when they have to be shipped. For roofs, wever, this construction is light and relatively cheap, as the total thickness ed not exceed 1¼ in for spans of 4 ft 10 in. To make the roof water-tight me water-proof covering is required. With a good coat of hard plaster or uged mortar on the under-side, the iron will not be affected by heat in case of e until a considerable time has elapsed; and even if the mortar on the under- ide should be more or less dislodged by the streams of water, it can be replaced,

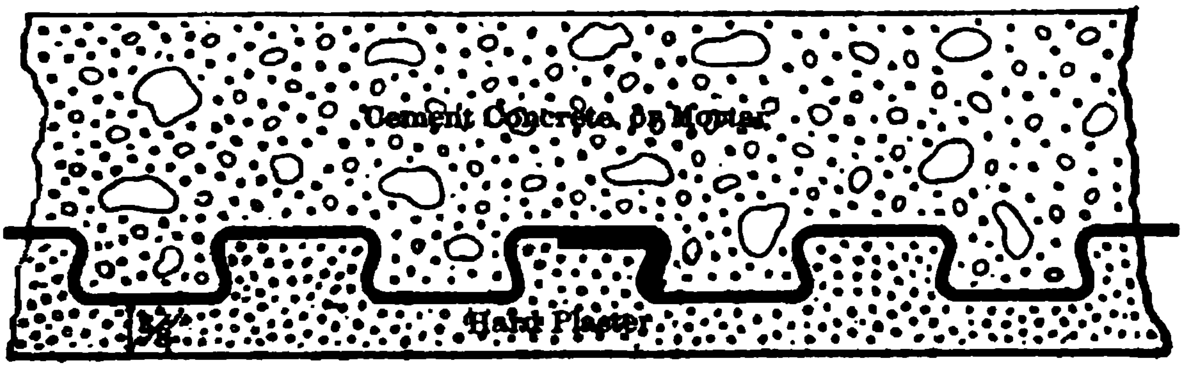


Fig. 36. Ferroinclave Reinforcement for Concrete Floors

t a very slight expense. Another advantage in the use of Ferroinclave for roofs that a building can be covered and made water-tight in the most severe winter eather and the cement applied during the following spring.

Ferroinclave is made in sheets 20 in wide and up to 10 ft long, and it is usu- lly of No. 24 gauge. For roofs it is attached to purlins in the same way that on roofing is attached, the most economical spacing of the purlins being 4 ft 10½ center to center, which accommodates sheets 10 ft long and leaves an end- p of 3 in. For the cement top coat on roofs, a mixture of one part Portland ment to two parts sand, applied to a thickness of ⅜ in above the top of the eets, is sufficient. For floors a rich gravel or crushed-stone concrete should be ed, the thickness being governed by the span and the loads to be supported.

The following table shows the ultimate strength of No. 24 Ferroinclave with fferent thicknesses of concrete, as determined by actual tests with sheets 20 in ide over a 4-ft 10½-in span:

hickness in inches of 1 : 2 mortar above the metal	1½	2	2½	3	3½	4
ltimate strength in lb per sq ft for a span 4 ft 10½ in	615	915	1220	1560	1860	2120

A factor of safety of 6 should be ample for ordinary loads.

Ferroinclave is especially adapted for the roofs and floors of large manufac- ring plants, and may be used to advantage for partitions, stair-treads, vats, ater-closet partitions, and fire-proof doors.

Berger's Multiplex Steel Plate. Fig. 37 shows a section of a corrug steel plate manufactured by the Berger Manufacturing Company, Can Ohio, for floor and roof-construction, the plate being an invention of G. Fugn As shown in the illustration, it consists of a series of vertical corrugation sheet steel, painted or galvanized, ending at the top and bottom in three l circle arches, separating the vertical sides of the corrugations from each o and giving stiffness to the top and bottom of the plate. The plate is made depths, *D*, of 2, 2½, 3, 3½, and 4 in, and in corresponding widths of 13½, 14½, and 15 in. The maximum length of plate is 10 ft. It can be made of gauge of steel, from No. 24 to No. 16, but No. 18 is as heavy a weight a generally required. For floors and roofs, the corrugated plate is laid on to the beams and the top portion filled with concrete and leveled off about above the plate. For wooden floors the nailing-strips may be embedd the concrete and the bottom of the strips raised only about ½ in abov top of the plate. The construction is very light and strong and require centering. It cannot be plastered, however, on the under side; and whe

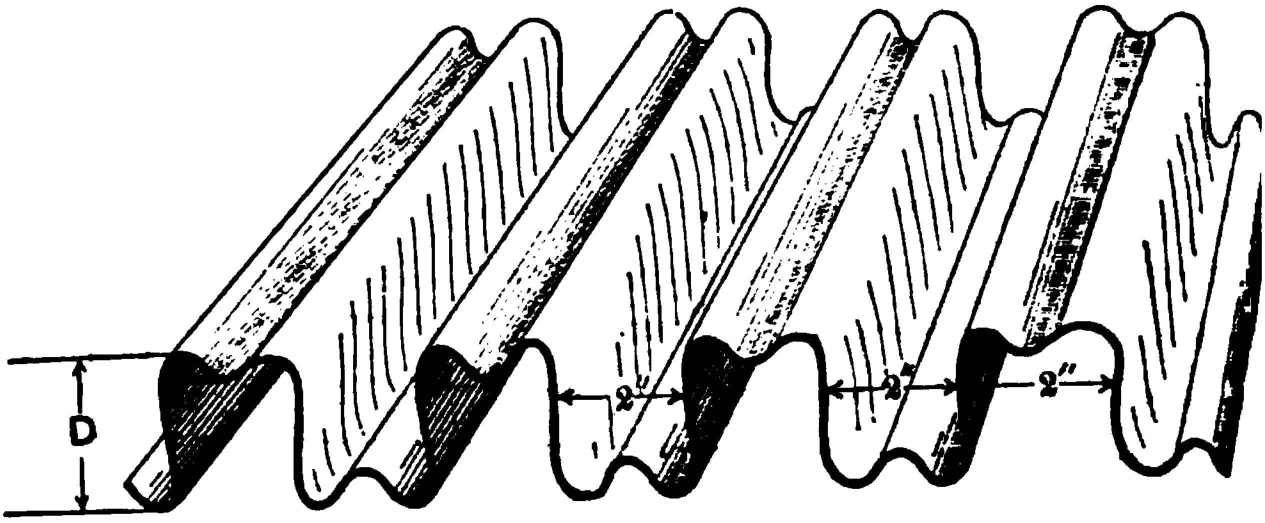


FIG. 37. Berger's Multiplex Steel Plate

plaster ceiling is required it must be constructed independently of the plat means of furring-strips and metal lath. The weight of the 4-in plate, wit 1 : 2 : 5 furnace-slag concrete leveled 1 in above the top of the plate, is ab 40 lb per sq ft, and the safe load for a 10-ft span is given at 270 lb per sq While this floor has several practical advantages, it cannot be consid thoroughly fire-proof, because the metal is exposed on the bottom. But a plastered ceiling on the under side, the iron would probably not be affe by any ordinary fire before the latter could be controlled.

Permanent Centering. Numerous forms of sheet-metal fabrics have b developed in recent years for use as floor-reinforcements. They consist, erally, of steel plates pressed into series of solid ribs, variously spaced, betw which the metal is stamped or perforated, or deployed into an open mesh-w The characteristic form is shown in Fig. 38. The mesh is kept small eno to prevent ordinary concrete from passing through. For use as a reinforcem the sheets are furnished either in flat or segmental form. A 1 : 2½ : 5 ston cinder concrete may be used, the thickness depending upon the span and the l to be provided for. For spans exceeding from 3 to 5 ft, according to the gaug metal, the sheets must be temporarily supported until the concrete has set. difficulty of providing efficient fire-protection on the underside of reinfo ments of this type, and around the lower flanges of the supporting steel be is a serious disadvantage. Besides, the bond between the metal and the f concrete is on one side of the sheet only. Some of the forms now on the mar

with their special characteristics, are briefly described in the following paragraphs.

Rib-Truss. These plates, manufactured by the Berger Manufacturing Company, Canton, Ohio, are designed with five longitudinal ribs, 6 in on centers, and ½, ¾, 1, and 1½ in high. The metal between the ribs is slit into trusses which are further reinforced with beads at right-angles to the main ribs. The standard sheets are 24 in in width and are carried in stock in lengths up to ft, and made of No. 24, 26, 27, and 28-gauge metal.

Self-Sentering. In this form, manufactured by the General Fireproofing Company, Youngstown, Ohio, the ribs are 1¾ in in height, 3⅝ in on centers, and connected by expanded metal. The sheets are 29 in in width and come in lengths from 4 to 12 ft, varying by units of 1 ft. Self-Sentering is made of Nos. 26, and 28-gauge metal. (See, also, page 885.)

Hy-Rib. Hy-rib metal, now controlled by the Truscon Steel Company, Youngstown, O., is made in sheets measuring 10½ in from center to center of side ribs and having four ribs 1³⁄₁₆ in in height; and also in sheets 14 in in width having three ribs. There is also a type known as the Deep Rib. The lengths are 6, 8, 10, and 12 ft. The sheets are of No. 24, 26, or 28 United States gauge, and are furnished either flat or in various types of curves. (See, also, page 886.)

Corr-Mesh. Corr-Mesh is manufactured by the Corrugated Bar Company, Inc., Buffalo, N. Y., which supplies, also, special clips for splicing and fastening the mesh to the supporting members. It is made in two types. One has ribs ¾ in high, spaced 3¾ in on centers; the other type has ribs ⁹⁄₁₆ in high, spaced on centers. For the ¾-in-rib Corr-Mesh the sheets are 13 in wide, and for ⁹⁄₁₆-in-rib Corr-Mesh they are 18 in wide. The mesh is furnished in United States standard gauges, Nos. 24, 26, and 28. Standard sheets are 6, 8, 10, and in length. No allowance need be made for side laps, but at least 2 in should allowed for end laps.

Duplex Self-Centering. The Sharon Steel Hoop Company, of Youngstown, Ohio, manufactures the Duplex Self-Centering. It is 23 in in width, is finished in lengths of from 4 to 12 ft, and in Nos. 24, 26, and 28 metal, United States gauge. It weighs 1.37 lb per sq ft for the No. 24 gauge, 1.03 lb for the 26 gauge, and 0.86 lb for the No. 28 gauge; and it has a corresponding cross-sectional area per foot of width, of 0.411, 0.308, and 0.257 sq in

Sectional Systems. During recent years, the UNIT SYSTEM OR SEPARATELY MOLDED SYSTEM, consisting of shop-made reinforced-concrete members, such as

girders, lintels, floor-slabs, and wall-panels, made at a factory and shipped to sites of building operations, has been receiving considerable attention in t country. This system is more completely discussed in Chapter XXIV, p 953, under the title Separately Molded Construction. Separately mol members have been used between the steel beams of fire-proof floor-construct as a substitute floor-filling for the usual terra-cotta or concrete floor-arc The advantages of such systems, where they are practicable, are obvious. S members are usually made as large as can be conveniently handled and of c paratively long span.

Disadvantages of Sectional Systems. The reason that the SECTION SYSTEMS have not found favor is because they necessitate a fairly uniform spac of beams throughout a structure, and this is generally impracticable. casting of the parts has hitherto not been commercially successful, as the for although used repeatedly, have been more expensive than the usual centel at the building; and it is also generally necessary to use a concrete that is ric and more carefully prepared in order that it may stand the additional handli Even with all possible care, the breakages in transportation are considera As the methods of manufacture of factory-made members are constantly be perfected, chiefly in mechanical contrivances for cheapening the forms and red ing the handling during the process of manufacture, the economy of this syst is being substantiated, and particularly when it is used in combination wit light structural-steel fire-proofed frame.

Waite's Concrete Beam. In Fig. 39 is shown a type of SECTIONAL FLO CONSTRUCTION that has been used in a number of buildings by the Stand

Fig. 39. Waite's Concrete I Beams

Concrete Steel Company of New York City. The floor-construction consists series of concrete I beams 10 or 12 in in depth, supported on the lower flange the steel beams, which are spaced from 5 to 7 ft apart. The concrete beams set about 18 in apart and the spaces between the lower flanges are filled in a cinder concrete of the same composition as the I beams. On the tops of concrete beams is placed a metal fabric of small mesh on which a lean-conc slab is laid. This makes a comparatively light floor-construction, becau the large spaces between the concrete beams. The concrete I beams are at the shop and allowed to harden before they are sent to the building. the lower flange is inserted, as shown, a steel reinforcement, of small circ or other cross-section, to furnish the necessary tensile strength. The be are cast with the proper lengths, in accordance with the drawings; and slight variations at the building are made up by filling the spaces between ends of the concrete beams and the webs of the steel beams, and covering webs of the latter with concrete. A similar construction, consisting of a s

beams, with lower flanges 1¼ in thick and 12 in wide and stems 2 in thick 12 in deep, of 1 : 4 cinder concrete, reinforced with ⁵⁄₁₆-in rods near the es, and without floor-finish of any kind, successfully withstood the fire, r, and load tests of the New York City Bureau of Buildings after having constructed 28 days. This system has proved to be practical in cases in h a flat or level ceiling is required and the steel floor-beams are 10 in or e in depth. The cost of construction compares favorably with that of r flush-ceiling types.

he Siegwart Floor System. This system (Fig. 40), designed by Hans wart, of Lucerne, Switzerland, is in extensive use in that country. The

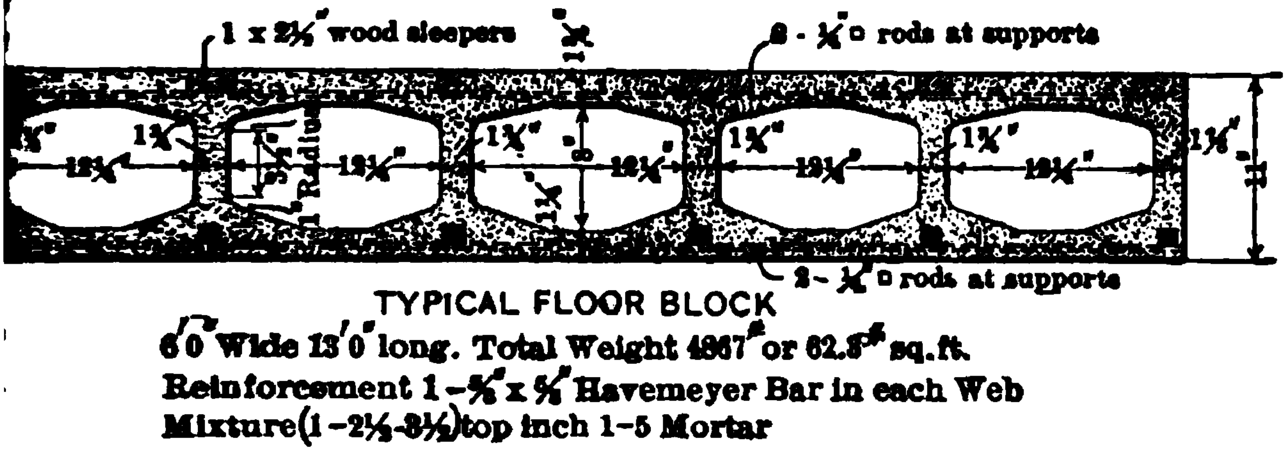


FIG. 40. Siegwart Reinforced-concrete Floor-construction

ional units are usually made 10 in in width, the height and reinforcement ying with the span and load. In a test on a beam of this type, designed to y a live load of 150 lb per superficial ft over a 16-ft span, the construction stood a satisfactory four-hour fire test with a load of 150 lb per sq ft, followed r the fire by a test with a load of 600 lb per sq ft. It is claimed for this em, that using the same working units for the strength of the material, the l weight of the construction is only one half that of a monolithic reinforced-rete floor designed to carry the same load with the same percentage of forcement. "The Siegwart Company claim their method to be much per than monolithic floors. From quotations furnished by their Canadian pany, the price in Montreal is quite a little less than the author's expe-e for monolithic floors in the same city, ranging from 17 to 26 cts per sq ft, ed for various spans and loads."* A modification of the Siegwart system been developed by Grosvenor Atterbury, and has been employed in two-story three-story residence-buildings for the Sage Foundation Homes Company rest Hills (Long Island), N. Y.

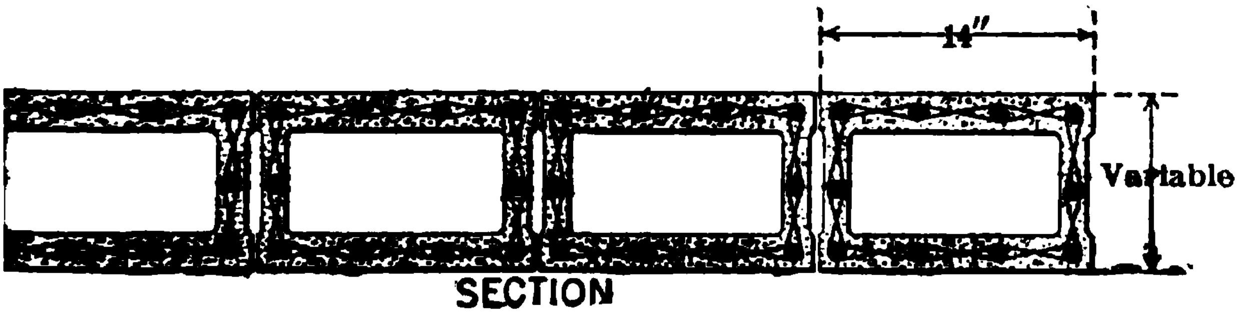


FIG. 41. Climax Reinforced-concrete Floor-construction

e Climax Floor System. This system (Fig. 41) was designed by S. M. lolph. The design is similar to that of the Siegwart floor system.

has. D. Watson, Concrete Construction with Separately Moulded Members and . Proc. Nat. Asso. Cement Users, Vol. VI, 1910.

The Vaughan Floor System. The Vaughan Company of Detroit, Mi is manufacturing a shop-made unit which is employed considerably throug the Middle West. The general form of this unit is like that of Waite's con beam, shown in Fig. 39.

The Watson Floor System. Two types of sectional floor systems for proof floor-fillings between steel beams are shown in Figs. 42 and 43. For

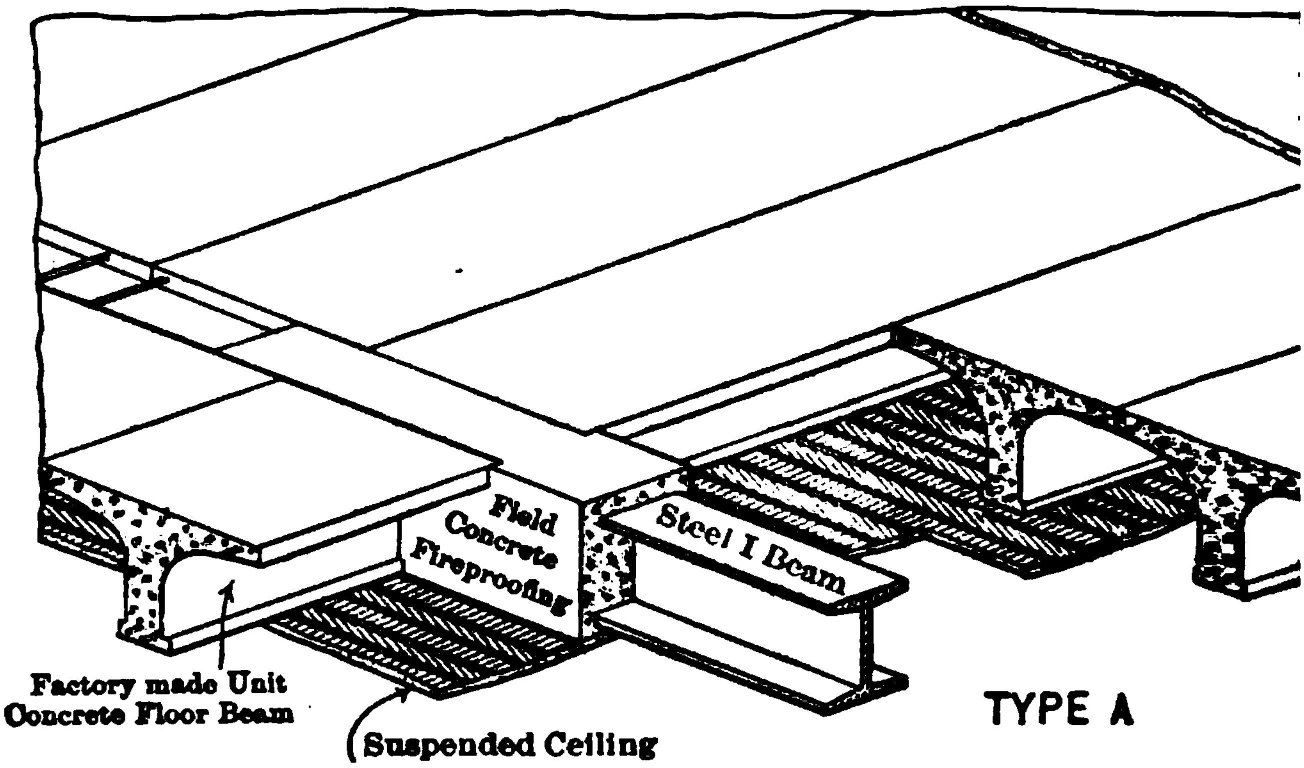


FIG. 42. Watson Reinforced-concrete Floor-construction. Without Slabs

spans and heavy loads, the T sections are used, laid side by side; and for s less than 20 ft and loads of 200 lb per sq ft or less, the beams are spaced on centers with flat slabs between. This system is controlled and installe the Unit Construction Company of St. Louis, Mo. Beams and girders

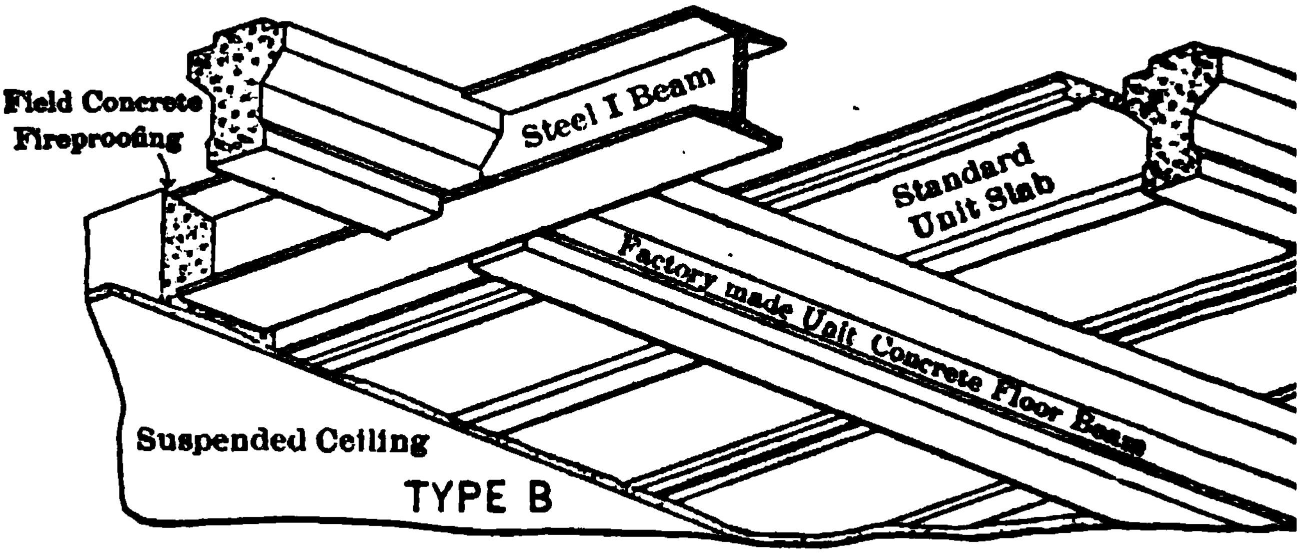


FIG. 43. Watson Reinforced-concrete Floor-construction. With Slabs

cast with unit frames in horizontal molds, and slabs are made on edge in forms. In the American School Board Journal for August, 1912, Theodo Skinner describes the construction and erection of a story-and-basement sc house with a structural-steel frame and shop-made reinforced-concrete j with unit-ribbed reinforced-concrete slabs.

Gypsum Floors. Gypsum has been extensively used for floors and ro fire-proof buildings. It furnishes a light construction which, with the additi

ntage of the rapidity with which it can be put in place, is economical not y with respect to the floor itself but also on account of a saving in the amount he structural steel supporting it. Another favorable feature is the great t-insulating property of gypsum, resulting in absence of condensation and a ction in the cost of heating the building.

he Metropolitan System. This construction consists of a series of steel les suspended from the supporting steel beams and encased in a slab of pure fined gypsum containing about 15% of wooden chips. The cables are generally posed of two No. 12 galvanized-steel wires, twisted. They are made contin- s over the supports, being securely fastened over the flanges of the end-beams hannels by heavy S hooks or other suitable means. The cables are spaced from 3 in apart, depending on the carrying capacity desired. They are held taut a ⅝-in round steel rod, laid at the middle of the span at right-angles to their ection. The mixture of gypsum and chips is sent to the work in bags and ed on wooden centers, as in the case of concrete floors, wet, and allowed to . The sides and flanges of the supporting steel beams are encased in the same terial, all as shown in Fig. 44. The minimum thickness of floor-slabs is 4 in;

F

Fig. 44. Metropolitan Fire-proof Floor

usual thicknesses for roofs are 3 and 3½ in The finished slabs weigh about per sq ft per in in thickness. Spans of from 6 ft 6 in to 7 ft are said to make most economical arrangement, all things considered. Spans should not eed 10 ft. The safe gross strength of the construction may be determined the formula,

$$w = \frac{24Td}{bL\sqrt{9L^2+d^2}},$$

hich

=the safe gross load per sq ft of floor or roof-surface, in pounds;

=the safe tensile strength of the twisted cables, in pounds, which for the ordinary case of two No. 12 cold-drawn steel wires, may be taken at 365 lb;

=the deflection of the cable, in inches, and equals the slab-thickness less the sum of the protection of the cables at the center of the slab and over the supports, that is, ordinarily, the slab-thickness less 1 in;

=the spacing of the cables, in inches;

=the span, or distance, between centers of supports of the slab, in feet.

o tie-rods are necessary in this floor-construction; but in the end-bays, when lateral stiffness of the beams together with the compressive strength of the -slab is not sufficient, struts must be provided of such size and spacing as necessary to resist the pull of the cables. This floor-construction is con- d and installed by the Keystone Gypsum Fireproofing Corporation, New

Metal Lumber. A system of pressed-steel I joists, channel-joists, co joists, wall-ribbons, etc., has been developed as a substitute for ordinary wo framing in the construction of walls, floors, roofs, and partitions. In the construction, the I joists and channel-joists, of from No. 16 to 12 United St gauge sheet metal, are braced by metal bridging, to give additional rigidity. typical floor-construction is shown in Fig. 45. The steel floor-joists are cov above with a concrete slab reinforced with expanded-metal lath, and the l flanges are protected by a ceiling of metal lath and cement plaster attache the joists by means of the prongs. The metal-lumber joists frame into ordi steel girders, resting on a shelf-angle as shown in the drawing. The joists cut to length at the factory, properly marked and tagged and, with the erect diagrams, are shipped to the site. All joints and splices are riveted in the f The steel girders should be properly incased in some fire-proof covering.

Fig. 45. Berger's Metal Lumber and Concrete Floor-construction

materials for this floor-construction are manufactured by the Berger Manu turing Company, Canton, Ohio; the General Fireproofing Company, You town, Ohio, the Truscon Steel Company, Youngstown, Ohio; and the Nati Pressed Steel Company, Massillon, Ohio. They publish safe-load tables metal-lumber I joists and channel-studs for spans of from 4 to 20 ft. This tem, contemplating the use of steel joists and girders, not thoroughly inc with fire-proof materials, cannot ordinarily be considered thoroughly resistant, although a specially constructed floor with all the steel covered protected with fire-proofing-material has passed the fire test prescribed by New York Building Code. (See page 827.) It has been extensively use replace combustible building-construction, especially in residence-buildings.

Protection of Girders and Beams. No form of floor-construction ca considered thoroughly fire-proof unless it includes a protection of the l flanges of all steel beams and girders, or provides for the protection of all used in its construction or support. The material used for the protective co ing is generally the same as that used in the floor-construction itself.

ncipal materials are tile, either dense, porous, or semiporous; gypsum; and ncrete, either of cinders, stone, or slag Beam-protection, where the floor-nstruction incases the side of the beams, as in Figs. 17, 19, or 34, should ver be less than 1 in thick Where paneled ceilings are used, that is, where e lower part of the beams is below the lower side of the floor-construction, in Figs. 18 or 30, the protection should be increased to at least 1½ in at all nts.

Tile Beam-Protection. When tile is used, there are two types of protec-n. In one case the blocks incasing the bottom flanges of the steel beams meet

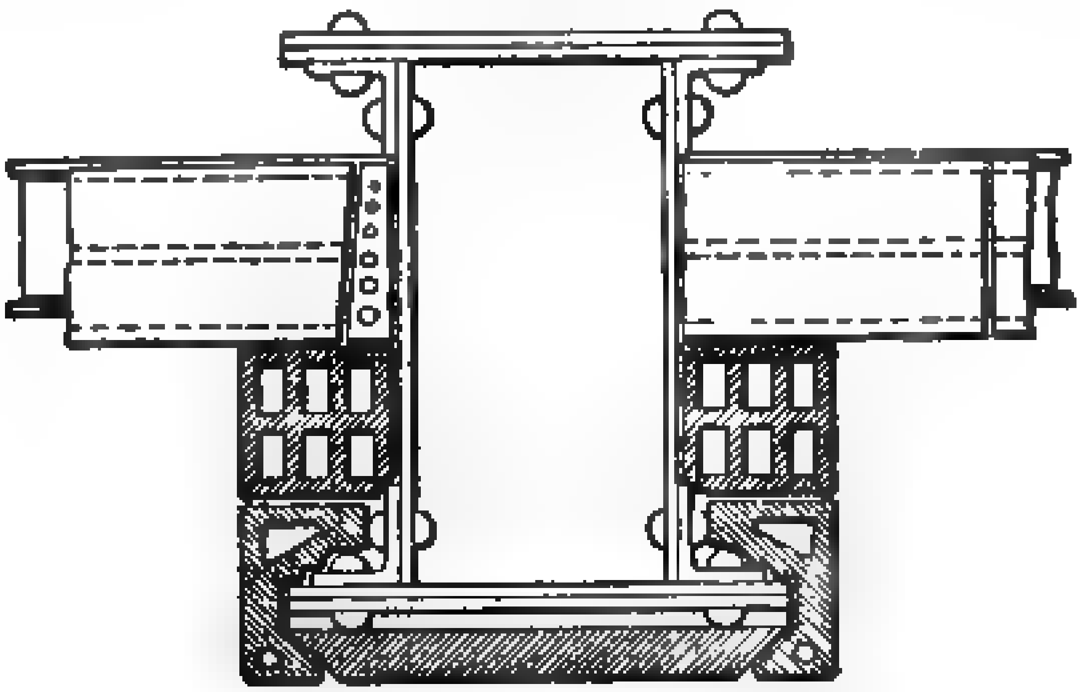


Fig. 46. Tile Protection for Box Girder

the middle of the lower side of the flanges; in the other, they simply turn er the edges of the bottom flanges and hold flat tiles with beveled edges

Fig. 47. Tile Protection for Single-beam Girder

at the lower side of those flanges. The latter is considered the better d, although in this method the danger of breakage of the part extending

under the flange is supplemented by the possibility of an omission of the protection-tiles. The blocks incasing the lower flanges may be the skewb of the arch, or they may be separate blocks. Different forms and condition illustrated in Figs. 15 to 24. Fig. 26 shows the entire beam protected by bl on both sides. Girders, which often project below the ceiling-line, are m more exposed to the effects of fire and water than the floor-beams, and should have, therefore, the most efficient protection. As a rule, such gi should be provided with not less than 4 in of terra-cotta protection at the s and 1½ in of solid tile on the lower side, with a space of ½ in between the t cotta tiles and the girder. Fig. 46 is a typical method of protecting girder means of hollow tiles. The bottoms of the skewbacks are prevented f spreading by wire ties placed in ~~the end-joints between~~ the soffit-tiles and ho into the round holes in the skewbacks. Single-beam girders are usually tected as shown in Figs. 22 and 47, the latter figure showing more particu the protection of a beam at the side of an opening in the floor.

Concrete Beam-Protection. A more thorough incasing of the webs lower flanges of beams and girders can be accomplished by the use of conc The superior fire-proof character of cinder concrete makes it the best mat for this purpose. If of sufficient thickness and properly applied, it will securely, without reinforcement, around the flanges of beams and gir But where it is less than 2 in thick, wire or metal lath, wrapped around flanges, should be embedded in it. A common form of concrete-protecti shown on right hand side of Fig. 30. Sometimes the soffit of the beam is tected by a concrete slab with an insulating air-space. This method is one may be advantageously used for the protection of girders. A fire test of form of girder-protection made in the Butterick Building, New York thoroughly established its efficiency. Hung ceilings are sometimes used a protection for the steel beams. This is very bad practice, as these cei are more than likely to collapse in a severe fire. The experience in the timore fire confirms this belief. (See, also, pages 780 to 782.)

The Fireproofing of Trusses. When steel trusses are used to suppor roof or several stories of a building it is very important that they be prote not only from heat sufficient to warp them, but also from expansion suffi to affect the vertical position of the columns on which they are supported. following description of the covering of the trusses in the Tremont Te Boston, Mass., furnishes a good illustration of the way in which this shou accomplished: "The steel girders were first placed in terra-cotta blocks o sides and below, these blocks being then strapped with iron all aroun girders, and upon this was stretched expanded-metal lathing, covered w heavy coating of Windsor cement; over this comes iron furring, which rec a second layer of expanded-metal lath, the latter, in turn, receiving the fin plaster. There is, consequently, in this arrangement for fire-protection, a dead-air space, then a layer of terra-cotta, a Windsor cement covering, an dead-air space, and finally, the external Windsor cement." Numerous s of terra-cotta tiles are made for incasing the structural shapes commonly in steel trusses. Some of these are shown in Fig. 48. The tiles should a be secured in place by metal clamps passing entirely around the envelo better still, by wrapping with wire lath. The tiling should then be plas with hard wall-plaster. Trusses, also, may be fire-proofed by complete casing the several members in cinder concrete, either with or without reinforcement. The method of incasing steel columns by means of the ce gun (page 826) is also applicable to the protection of steel trusses, and if o ficient thickness would probably serve as a suitable fire-protection; b

ble data on this latter point are as yet available. When trusses are to be proofed, the additional weight must be provided for in the strength of the es themselves.

SECTION OF BRACING

SECTION OF STRUT

FIG. 48. Tile Protection for Members of Steel Trusses

Steel Framing for Fire-proof Floors. Before the framing-plans of a build- can be made, it is necessary to decide, in a general way, upon the SYSTEM FLOOR-CONSTRUCTION or fireproofing that will be employed. Thus, if any one the LONG-SPAN SYSTEMS, such as the Herculean, Johnson, and many of the crete systems, is to be adopted, the girders should be spaced so that the r-construction will span between them, without floor-beams, as shown Fig. 49, while if an ORDINARY FLAT-TILE ARCH is to be used, floor-beams will

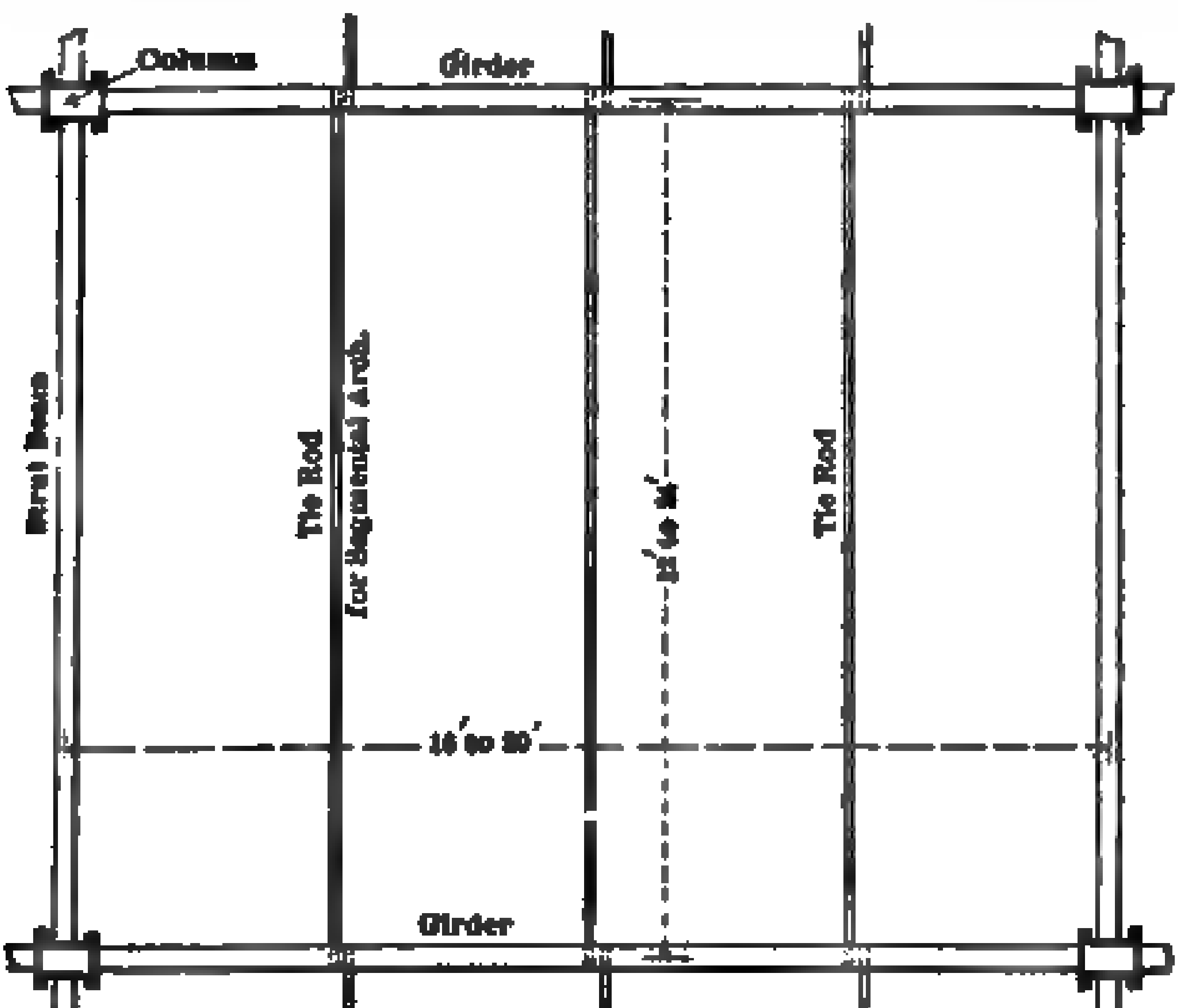


Fig. 49. Steel Floor-framing for Long-span Construction

be required, spaced from 5¼ to 9 ft apart, and these beams must be suppo
by girders, as indicated in Fig. 50. When there are no floor-beams, a stru
BEAM should be riveted between the columns, as in Fig. 49, to hold the la
in place during erection and to stiffen the building. It should be remembe
that with floor-beams spaced not more than 7 ft on centers, almost any sys

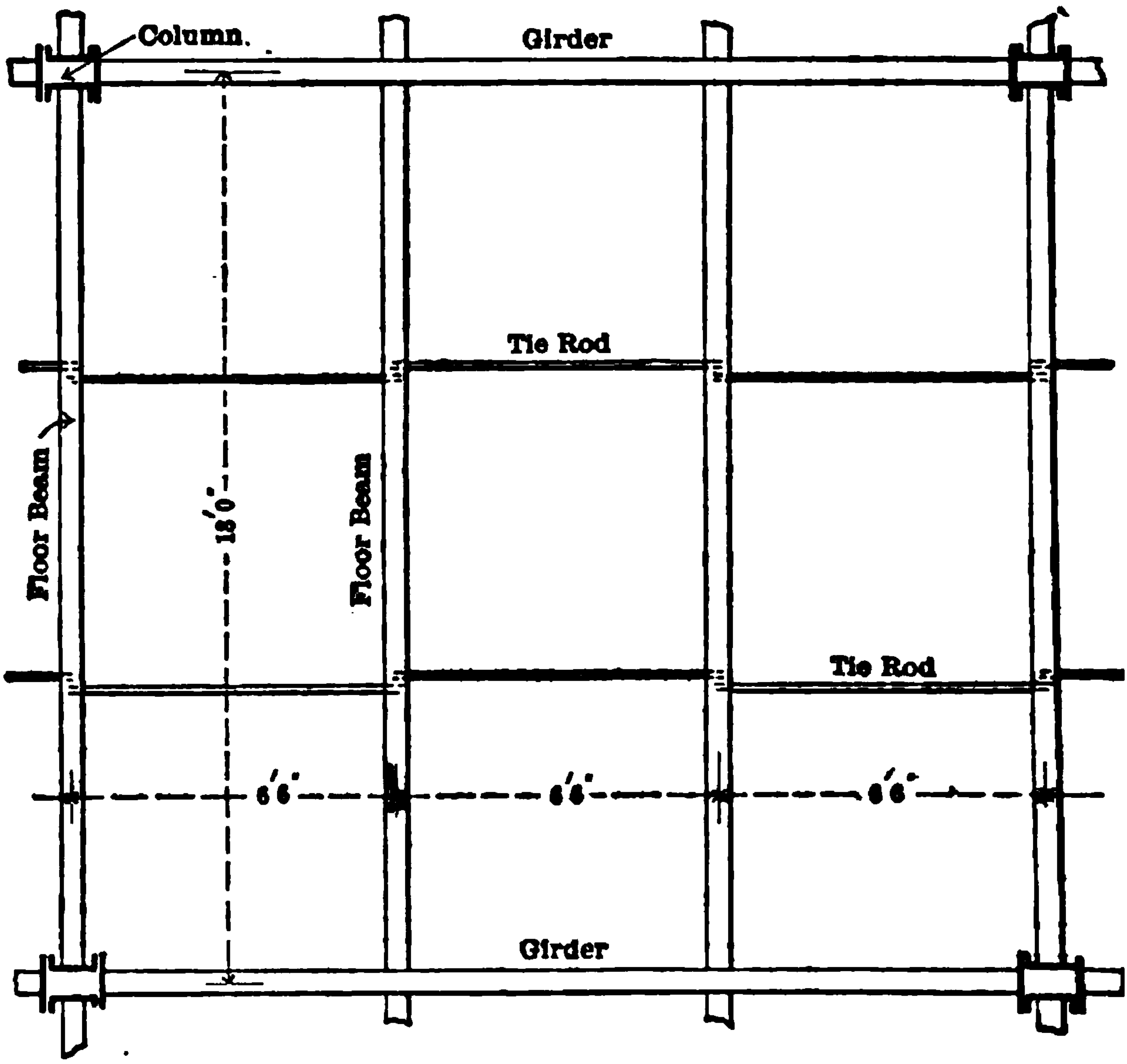


Fig. 50. Steel Floor-framing for Short-span Construction

of floor-construction may be employed; while if the floor-beams are omi
there are few systems to select from. With any form of filling between b
or girders, less steel is required for moderate than for excessive spans of b
or girders.

Computations for the Steel Framing. The computations for the
beams and girders of a fire-proof floor are very much the same as for a wo
floor. The load or loads which any given beam is required to support are
estimated and then the beam of the necessary size to support the load is sele
The DEAD LOAD for any fire-proof floor may be estimated with sufficient accu
by means of the data given in this chapter in connection with the diff
systems of floor-construction. The dead load should include the weight o
beams, the fireproofing, including all concrete filling, the plastering, fu
lathing, nailing-strips, and flooring. The LIVE LOADS may be estimate
means of the data given in Chapter XXI, pages 718 to 721.

Example. The best arrangement for the columns in a retail store is 1
them 18 ft on centers in one direction and 19 ft 6 in in the other. It is de
to run the girders as shown by Fig. 50, and to put a beam opposite each co

d two beams between the columns. It is required to determine the proper es of the beams and girders, using an ordinary end-arch construction between e beams

Solution. From Table VII, page 836, we find that the least depth of arch hich it is advisable to use is 10 in, but as we will probably have to use 12-in eams it will be better to figure on a 12-in arch, as this will give less filling n top. The weight of the 12-in arch will be about 39 lb per sq ft. We shall obably require 2 in of concrete filling on top, which will weigh 16 lb, and ½ in of light filling between nailing-strips, weighing, say, 9 lb per sq ft. The oring and nailing-strips will weigh about 4 lb, the plastering on the ceiling lb, and we must allow at least 6 lb per sq ft for the weight of the beams themves. These make a total dead weight of 79 lb per sq ft. The live load for retail store should be taken at 150 lb per sq ft, making a total load per square ot on the beams of 229 lb. The total load that each beam must be capable of pporting will be 6½ ft by 18 ft by 229 lb, or 26 793 lb, or 13.4 tons, which is umed to be uniformly distributed. From Table IV, page 580, we find that is load, with a span of 18 ft, will require either a 12-in, 45-lb beam, or a 15-in, 2.9-lb beam. The latter will be both stronger and cheaper, but will increase e thickness of the floor by 3 in and require additional filling.

The girder must support two concentrated loads of 26 793 lb or 13.4 tons ch. On page 566 it is stated that when a beam supports two equal loads plied at points one-third the length of the span from each end, the equivalent iformly distributed load may be found by multiplying one load by 2⅔. ultiplying 26 793 lb by 2⅔ we have 71 448 lb as the equivalent distributed d on the girder, to which should be added the weight of the girder. This quires a standard 24-in 79.9-lb beam (Table IV, page 577).

If instead of using tile arches between beams 6½ ft apart, we conclude to e the Herculean or Johnson construction spanning from girder to girder, we uld frame our floor as in Fig. 49. For this span we should require 10-in tiles, ighing 55 lb per sq ft. Allowing 8 lb for 1 in of concrete, 9 lb for filling, 4 lb flooring and strips, and 5 lb for plastering, we have 81 lb as the dead load per are foot. We have added nothing for the weight of the girder, as this will fully offset by the portions of the floor not loaded. The live load per square t will be 150 lb as before, and the total load to be supported by the girder, ft by 19 ft 6 in by 231 lb, or 81 081 lb, or 40.54 tons, which will require a in 79.9-lb beam (Table IV, page 577). Hence by this arrangement we save weight of the floor-beams; but a 6-in strut-beam should be placed between columns, as in Fig. 49. The calculations for any other floor-construction similar to the calculations for this example, the only variations being in figuring of the dead weights of the construction.

ables for Floor-Beams. It is a difficult matter to prepare tables that may enerally used, showing the size of steel beams required for fire-proof floors, use such beams are often irregularly spaced, and there is a wide variation e dead loads. The following tables, however, may be used in making approxe estimates and in checking the computations for any particular floor. The of I beams given may be safely used where the total live and dead loads ot exceed the values given in the headings. The total loads should include ient allowance for the weights of any partitions that the floor-beams may lled upon to support.

ble XIII gives the sizes and weights of I beams for floors of offices, s, and apartment houses; Table XIV, for floors of retail stores and asly-rooms; and Table XV, for floors of warehouses. The total loads used computations are, respectively, 120, 200, and 270 lb per sq ft.

Table XIII. Sizes and Weights of I Beams for Floors of Offices, Hot and Apartment-Houses

Total load, 120 pounds per square foot

Span of beams in feet	Distance between centers of beams in feet									
	4½		5		5½		6		7	
	in	lb	in	lb	in	lb	in	lb	in	lb
10	6	12¼	6	12¼	6	12¼	6	12¼	7	15
11	6	12¼	6	12¼	7	15	7	15	7	15
12	6	12¼	7	15	7	15	7	15	8	18
13	7	15	7	15	7	15	8	18	8	18
14	7	15	8	18	8	18	8	18	9	21
15	8	18	8	18	8	18	9	21	9	21
16	8	18	9	21	9	21	9	21	10	25
17	9	21	9	21	9	21	10	25	10	25
18	9	21	9	21	10	25	10	25	12	31[illegible]
19	9	21	10	25	10	25	10	25	12	31[illegible]
20	10	25	10	25	12	31½	12	31½	12	31[illegible]
21	10	25	12	31½	12	31½	12	31½	12	31[illegible]
22	10	25	12	31½	12	31½	12	31½	15	42
23	12	31½	12	31½	12	31½	12	31½	15	42
24	12	31½	12	31½	12	31½	15	42	15	42
25	12	31½	12	31½	15	42	15	42	15	42

Table XIV. Sizes and Weights of I Beams for Floors of Retail Stor and Assembly-Rooms

Total load, 200 pounds per square foot

Span of beams in feet	Distance between centers of beams in feet									
	4½		5		5½		6		7	
	in	lb	in	lb	in	lb	in	lb	in	
10	7	15	7	15	7	15	8	18	8	[illegible]
11	7	15	8	18	8	18	8	18	9	[illegible]
12	8	18	8	18	9	21	9	21	9	[illegible]
13	8	18	9	21	9	21	10	25	10	[illegible]
14	9	21	9	21	10	25	10	25	12	[illegible]
15	9	21	10	25	10	25	12	31½	12	[illegible]
16	10	25	10	25	12	31½	12	31½	12	[illegible]
17	10	25	12	31½	12	31½	12	31½	12	[illegible]
18	12	31½	12	31½	12	31½	12	40	12	[illegible]
19	12	31½	12	31½	12	40	12	40	15	[illegible]
20	12	31½	12	40	12	40	15	42	15	[illegible]

Table XV. Sizes and Weights of I Beams for Floors of Warehouses

Total load, 270 pounds per square foot

an of eams n feet	Distance between centers of beams in feet									
	4½		5		5½		6		6½	
	in	lb	in	lb	in	lb	in	lb	in	lb
10	8	18	8	18	8	18	9	21	9	21
11	8	18	9	21	9	21	9	21	10	25
12	9	21	9	21	10	25	10	25	10	25
13	10	25	10	25	10	25	12	31½	12	31½
14	10	25	12	31½	12	31½	12	31½	12	31½
15	12	31½	12	31½	12	31½	12	31½	12	40
16	12	31½	12	31½	12	31½	12	40	12	40
17	12	31½	12	40	12	40	12	40	15	42
18	12	40	12	40	15	42	15	42	15	42
19	12	40	15	42	15	42	15	42	15	42
20	15	42	15	42	15	42	15	45	15	55

e-Rods. In all segmental arches and other types in which a thrust is ted against the beams, TIE-RODS must be provided to prevent the beams being pushed apart, and especially to prevent the outer bays from spread- They should run from beam to beam from one end of the floor to the . If the outer arches spring from an angle, as in Fig. 14, the tie-rods in bay should be anchored into the walls with large plate-washers. The ds should be located in the LINES OF THRUST of the arches, which are arly below the half-depth of the beams, and in some cases near the bottom es. If their appearance is objectionable, they should be hidden by a hung g. For constructional purposes they are desirable in all types of floor-ruction, even though the floors do not exert a thrust on the beams. rule tie-rods are proportioned and spaced according to some RULE OF B rather than by actual calculations of the thrust. For the interior arches ractice is probably safe enough, but for outside spans, and particularly gmental arches, the thrusts of the arches should be computed and the proportioned accordingly. The spacing of the rods is generally eight the depth of the supporting beams, but never more than 8 ft. For interior e arches, the following rule can usually be safely followed: for spans of less, use ¾-in rods spaced about 5 ft apart; for 7-ft spans, ⅞-in rods, art; and for 9-ft spans, ⅞-in rods, 4 ft apart.

HORIZONTAL THRUST of an arch may be found by the following formula:

$$T = \frac{3\,wL^2}{2\,R}$$

in which

T = pressure or thrust in pounds per linear foot of arch;
w = load on arch in pounds per square foot, uniformly distributed;
L = span of arch, in feet;
R = rise of segmental arch, or effective rise of flat arch, in inches.

The RISE of a segmental arch is measured from the springing-line to the so of the arch at the middle. For flat hollow-tile arches, the effective rise may figured from the top of the beam-flange to the top of the tiles. As the ti usually project from 1½ to 2 in below the bottom of the beams, the effecti rise will be from 2 to 2½ in less than the thickness of the arch. For the inter arches of a floor, w may be taken for the live load only, but for the exter arches, w should include both the full dead and live loads. Having found thrust of the arch, the SPACING OF THE RODS of any particular size may readily determined by dividing the safe load given for that size of rod in table on page 388, allowing 16 000 lb UNIT STRESS, by the thrust. The res will be the spacing in feet.

Example. What size of tie-rods and what spacing should be used for floor-construction described on page 863, in the preceding example?

Solution. The depth of a tile arch is 12 in, the dead load 79 lb and assumed live load 150 lb. The span between the beams is 6½ ft. Then, the interior arches, $w = 150$ lb, $R = 12 - 2\frac{1}{2} = 9\frac{1}{2}$ in, $L = 6\frac{1}{2}$ ft and T $(3 \times 150 \times 42.25)/(2 \times 9\frac{1}{2}) = 1\,000$ lb. The tensile strength of a ¾-in rod, upset, at 16 000 lb per sq in, is, from Table II, page 388, 4 832 lb. Div ing this by 1 000 we have a little less than 4 ft 10 in as the spacing. T tensile strength of a ⅞-in rod is given as 6 720 lb, which would admit of a sp ing of a little more than 6 ft 8 in. For the outer spans, w should be taken $150 + 79 = 229$ lb. Then $T = (3 \times 229 \times 42.25)/(2 \times 9\frac{1}{2}) = 1\,526$ lb. this thrust we should use ⅞-in rods spaced about 4 ft 5 in apart.

Load-Tests. It may be desirable at times to test fire-proof floors after th have been installed. The same precautions should be taken as for tests on re forced-concrete construction, described on page 967. If it is desired to determ from such tests the ULTIMATE STRENGTH, a section of the floor of a width eq to the span should be cut loose from the rest and loaded to destruction, supporting steel beams being shored up during the test. The SAFE WORK LOAD is found by dividing the BREAKING-LOAD by the proper FACTOR OF SAFE

5. Fire-proof Roof-Construction

Flat Roofs. Flat roofs are constructed in the same way as the floors, exc that the beams and girders are set so as to give a slight pitch to the roo drain the water. As the ROOF-LOADS are usually less than the FLOOR-LOADS as there are no partitions to be supported, the arches or roof-panels are usu considerably lighter than the floor-panels, but the general construction is pr tically the same for both. When the roof is formed of reinforced conc the beams may be set so that the concrete will give the desired inclina to the roof and will have a nearly uniform thickness, as this reduces the am of concrete required, and also the weight. In cases where level ceilings are sired, however, it would be cheaper to set the roof-beams level and to gi the roof with dry cinders, as the cost of the hung ceiling would more than ci the cost of the extra construction necessary to take the added weight of cin fill. If the roof is to be covered with tin or copper, nailing-strips should be bedded in the concrete, as for wooden floors, and the entire roof sheathed. is claimed that tin or copper laid over terra-cotta or concrete will rust out few years.* Gravel or tile roofs may be built without woodwork of any k Whether terra-cotta, gypsum tile, or concrete is used for the roof-panels, sides and bottoms of the steel beams and girders should be efficiently prote

* Freitag.

well as all columns and all other structural metal in the roof-space. In ordinary building, in which there are stair-wells or elevator-wells, the roof d upper ceiling are likely to be more severely tested by heat, in case of fire, an any of the floors below, and experience has shown that this part of the build- g often has the poorest protection.

Pitched Roofs. Pitched roofs may be constructed in various ways, accord- g to the material that is to be used and the kind of roofing that is to be em- oyed. When terra-cotta or gypsum tile is to be used for the fireproofing, the st common method of construction is that which involves the framing of the of with I-beam rafters and T-iron purlins, set horizontally and spaced 1 in ther apart than the lengths of the tile. Between the tees, book tiles, or roofing- ts are placed as in Fig. 51, and the roofing is applied directly to the surface of

Fig. 51. Tile Fireproofing for Roof-construction

e tiles. If the roofing is to be of slate or of clay tiles, solid, porous terra-cotta cks should be used between the tees, as nails are held better by solid blocks an by hollow tiles; gypsum roof tile is also suitable for this purpose The ne construction may be used for flat roofs; but on account of the expense of e tees it will usually be more expensive than the construction above described, d not as strong or desirable. With the construction shown in Fig. 51, it is possible, by any economical method, to efficiently protect the bottom of the irons from the effects of heat. Reinforced-cinder concrete, or reinforced ous terra-cotta tile, Johnson System, affords an excellent and also an nomical construction for fire-proof pitched roofs. Either of these construc- ns may be filled between or on top of the rafters without the use of purlins, ept about once in from 6 to 10 ft, to prevent sliding and to stiffen the roof. hree-inch plates of concrete, with expanded metal embedded, have been essfully used in spans of from 6 to 7 ft and in some cases even in 8-ft spans. e concrete is deposited on wooden centerings, as in the floor-construction, . the upper side is smoothed off during the setting and floated smooth and ight to receive the roof-covering"* The roof-covering, usually slate, or

* Freitag.

clay tiles, may be nailed directly to the concrete, as nails are held nearly well by cinder concrete as by wood. This applies only to cinder concrete, as is quite impossible to nail into rock concrete or gravel concrete. In concr roofs the rafters, also, should be surrounded with concrete held in place

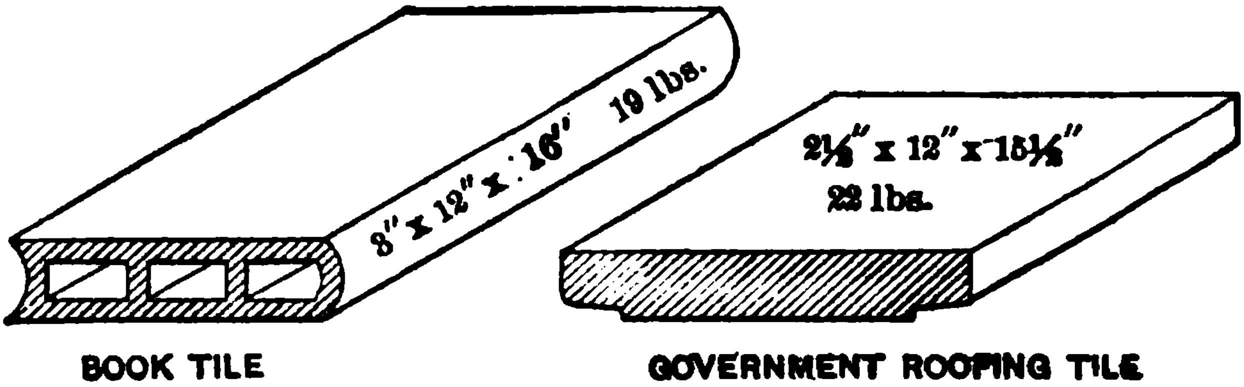


FIG. 52. Hollow Book Tile and Solid Tile for Roofs

metal lath. With terra-cotta roofs, the beams should be incased with ter cotta blocks. Fig. 52 shows the standard shapes of book tiles and solid roofi tiles. These are made 2, 2½, and 3 in thick, and from 16 to 24 in long. Thr inch book tiles weigh about 13 lb per sq ft, and 2½-in solid tiles about 16

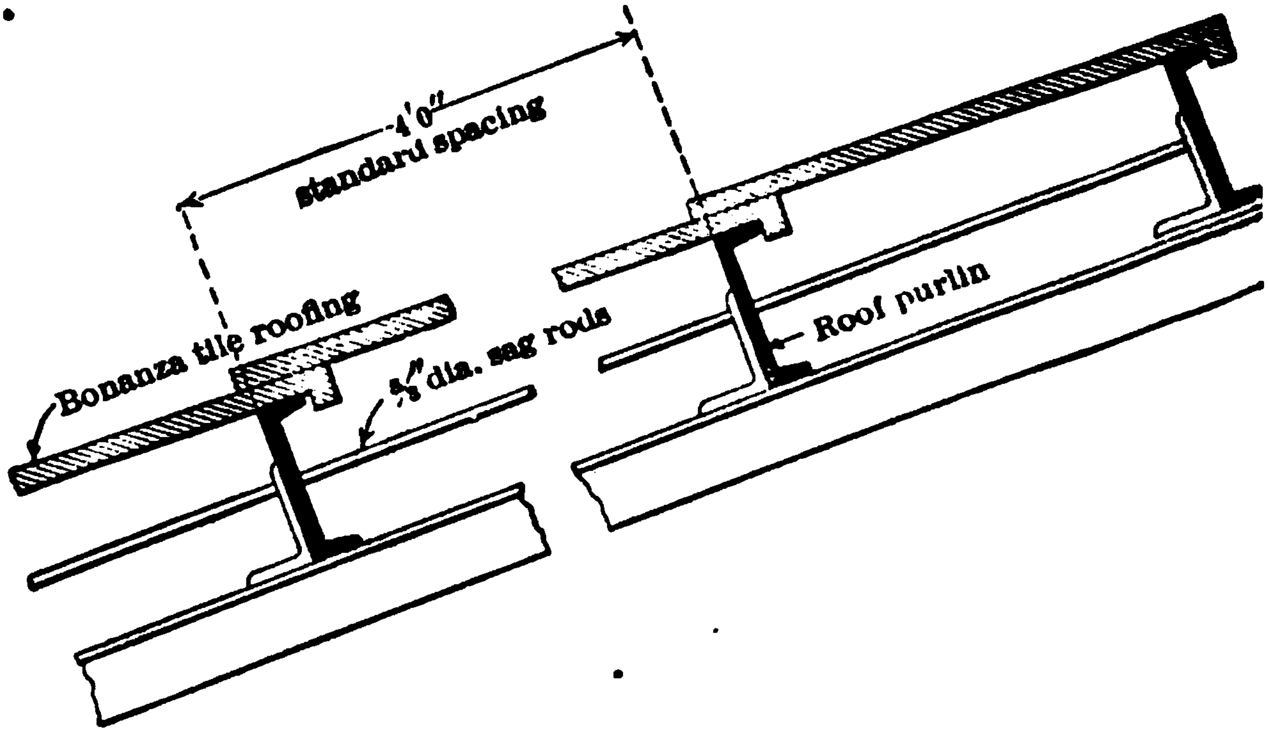


FIG. 53. Bonanza Reinforced-cement Tiles for Pitched Roofs

per sq ft. Tiles of both of these shapes are also used for ceilings and whe light, fire-proof filling is required.

Reinforced-Cement Tiles. Cement tiles of interlocking types, made in factory and reinforced with metal fabric or mesh, may be laid without sheatl directly on steel purlins. This type of construction, however, is suitable as a semifire-resisting roof-covering, as it is usually made with plates of ins cient thickness and does not contemplate the thorough incasing of the s understructure with concrete or other fire-resisting materials. Bonanza Cem Tile roofing is a type of this shop-made tile and is manufactured and contro by the American Cement Tile Manufacturing Company, Pittsburgh, Two types of tiles are made, one for pitched-roof and the other for flat- construction (Figs. 53 and 54). The properties of the tiles are given in the lowing tabulation:

Standard, Pitched-Roof Tiles

Thickness of tiles	about 1 in
Over-all dimensions of tiles	26 by 52 in
Tile-surface exposed to weather	24 by 48 in
Number of tiles per 100 sq ft of roof	12½
Weight of tiles per 100 sq ft of roof	1450 lb

Standard, Flat-Roof Tiles

Width of tiles	24 in
Length of tiles	60 in or less
Thickness of tiles	1½ in
Weight of tile-construction	16 lb per sq ft

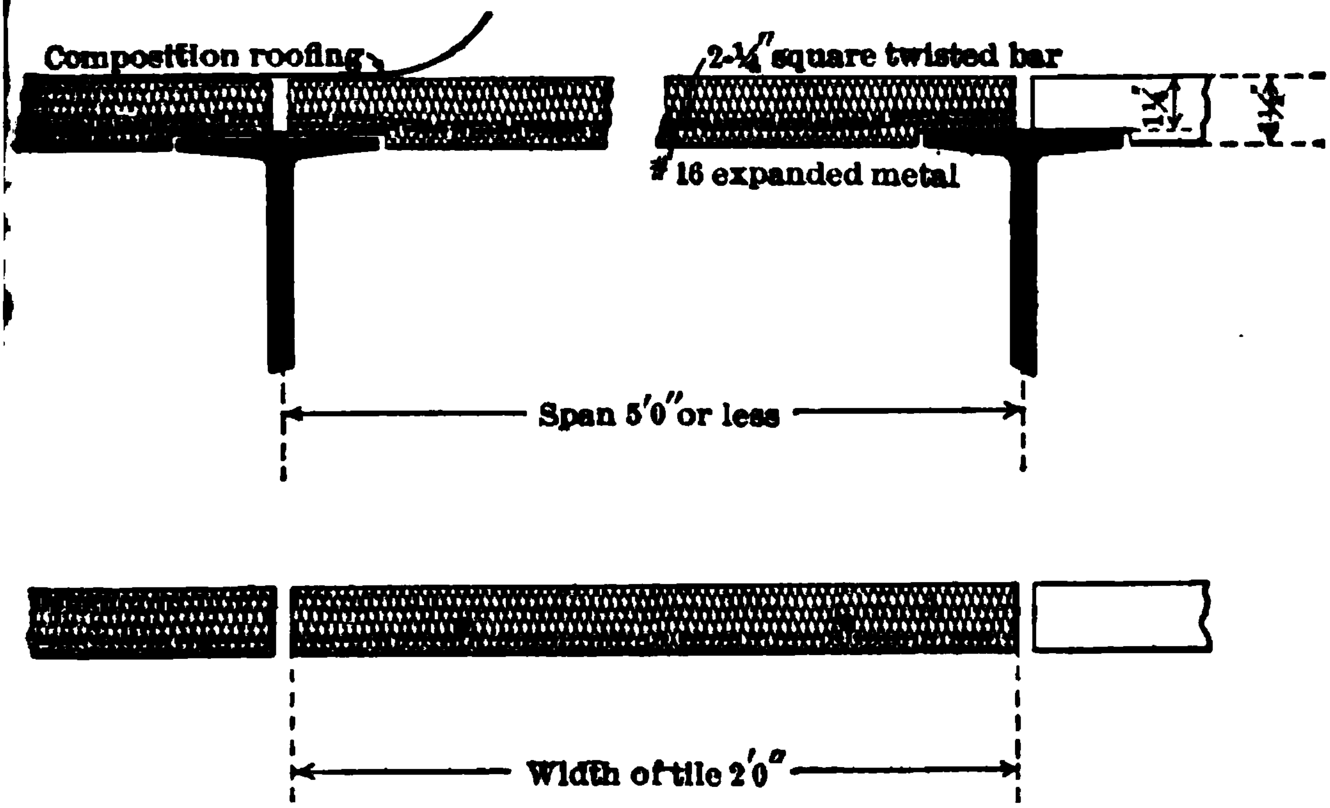


Fig. 54. Bonanza Reinforced-cement Tiles for Flat Roofs

The flat-roof tiles are designed for and have been used in connection with buildings for manufacturing-plants on spans of 5 ft between purlins. On these spans they have been tested up to an ultimate live load of 250 lb per sq ft. The top surfaces of these tiles are finished in a weather-proof and water-proof material of a dark, terra-cotta-red color.

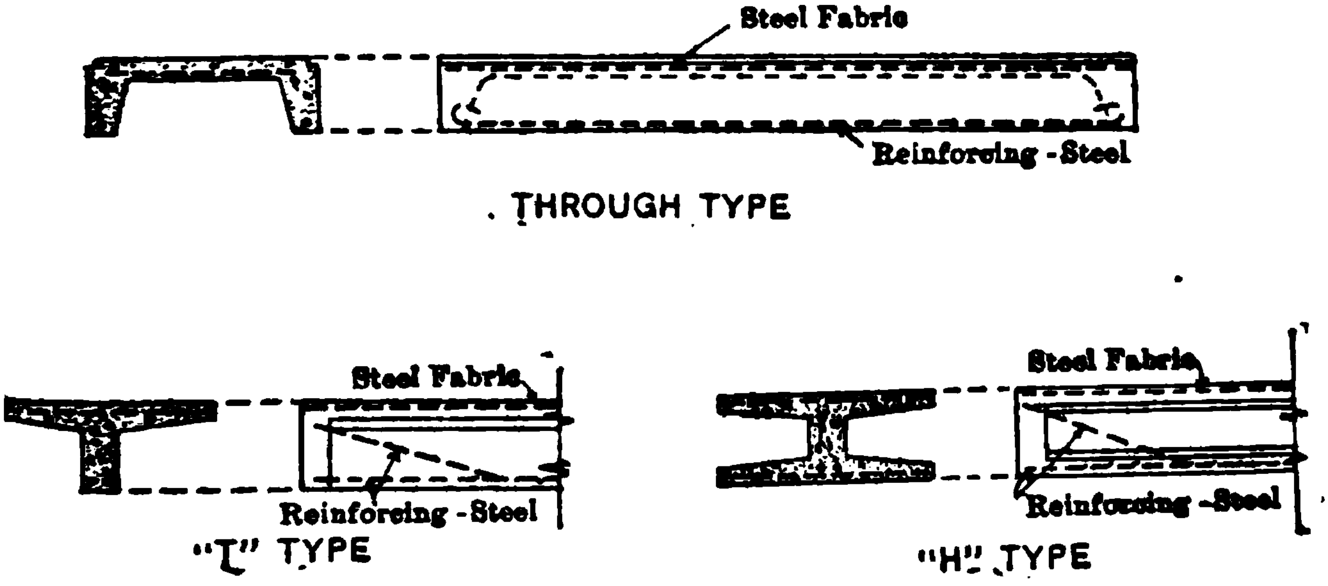


Fig. 55. Structolite Roof-tile

Structolite Roof-Tile. Shop-made roof-tiles made of a dense quick-setti gypsum cement, called STRUCTOLITE by the manufacturers, are put on the mark by the United States Gypsum Company of Chicago, Ill. The material used said to have an average ultimate crushing strength of 2000 lb per sq in. As tl material weighs only 77 lb per cu ft, a very light roof-construction results. Tl tiles are reinforced with steel in much the same manner as reinforced concret and their strengths are figured by the same formulas, using working stress appropriate to the materials. For spans from 4 to 6 ft, a trough-like tile is use as shown in Fig. 55. For greater spans, up to 10 ft, the T TYPE and H TYPE tile are used, the latter when a continuous flat ceiling is desired. The tiles a placed directly on channel or I-beam purlins, but when the flanges of the purlii are less than 2½ in wide, bearing-plates should be inserted between the tiles ar purlins. The weights of the roof-tiles in lb per sq ft are as follows, the til themselves being generally designed for safe superimposed loads of 50 lb p sq ft.

Span, ft	Depth, in	Trough type, lb	T type, lb	H type, lb
4	5	14	....	....
5	5	14	....	....
6	5	15	13½	18½
7	6	....	14½	21
8	6	....	14½	21
9	7	....	16	22
10	7	....	16	22

Robertson Process. Under the name of ROBERTSON PROCESS FLOO the H. H. Robertson Company, Pittsburgh, Pa., make and install gypsum floo construction of the same general character and design as the Metropolitan Floo (page 857). They also manufacture pre-cast roof-tiles designed on the sam suspension-principle. The cables protrude about 2 in at the ends of the slab near the top surface. When set in place on the roof-purlins with their en abutting, the projecting ends of the cables of adjoining slabs are tied togeth by a device that draws them taut, thus effecting continuity. The tiles are ral betted at the ends where the cables emerge, and these rabbets are filled wit gypsum, covering over and protecting the cable-connections. The following a the standard sizes of roof-tiles:

3 in thick, 24 in wide, varying in length by 3 in, from 4 ft 0 in to 6 ft 0 in
3 in thick, 21 in wide, varying in length by 3 in, from 6 ft 0 in to 6 ft 9 in
3 in thick, 18 in wide, varying in length by 3 in, from 6 ft 9 in to 7 ft 0 in
3½ in thick, 15 in wide, varying in length by 3 in, from 7 ft 0 in to 8 ft 0 in
3½ in thick, 12 in wide, varying in length by 3 in, from 8 ft 0 in to 8 ft 6 in

The weight per sq ft is 14 lb for the 3-in tiles and 16 lb for the 3½-in tiles.

Mansard Roofs are usually framed with rafters, riveted or bolted to wal plates. The space between the rafters may be filled with cinder concrete, holl partition-tiles, or blocks extending from rafter to rafter, as in Fig. 56. Slat or tiles may be nailed directly to cinder concrete or to porous terra-cotta. Prol ably the best way to attach slates or tiles is to nail 1¼ by 2-in wooden strips t the outer face of the concrete or terra-cotta, set them at the proper distanc apart to receive the slates or tiles, and then plaster between the strips wit

...nt mortar. This gives a better nailing for the roofing, and the wooden ...s are not affected by fire until the slate is practically destroyed.

...oof-Coverings. The materials ordi...ly used for the roof-covering of fire...of buildings are: (1) tar and gravel; ...asphalt and gravel or sand; (3) vitrified ...a, bricks, or slate tiles over tarred felt. ...r and gravel, or asphalt felting and ...el or sand, offer the cheapest roof ...able for a fire-proof building; and ...n a good quality of felt and distilled ...h or the best grades of asphalt are ...d, make a very satisfactory covering. ...h roofs, however, require to be renewed ...ut every ten years. The roofing is put ... in the same manner as over wooden ...truction, the felt being laid directly on ... concrete. Probably the best flat roof ...t can be put on a building is one of ...ified or slate tiles, laid over five plys of ...ed felt. The felt is laid and mopped ... for a gravel roof, and the tiles are ...dded on the felt in cement mortar. Vitrified tiles, about 8 in square and ... in thick, are made for this purpose, and slate tiles, 12 in square by 1 in ...k. have been used. Flat, vitrified-brick tiles, also, are used. Gravel roofing ...uld not be used on roofs which have an inclination exceeding ¾ in in 1 ft. ...r pitched or inclined roofs, slates, clay tiles, or metal tiles may be used. ...y tiles are superior to slate when exposed to fire and are generally to be ...ferred to slate; this is especially true of some of the patent interlocking tiles. ...e, also, pages 1582 to 1587, and 1595 to 1599.)

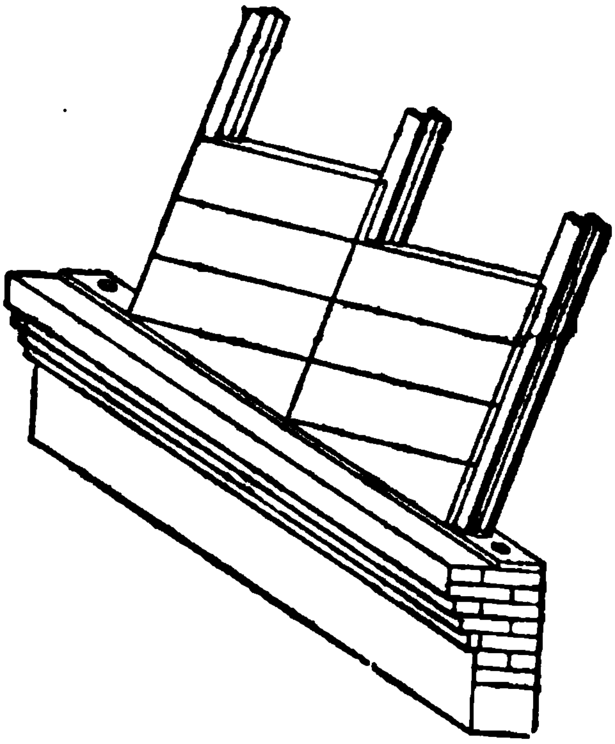


Fig. 56. Tiles for Mansard Roof

Suspended Ceilings. Office-buildings, apartment-houses, etc., having flat ...s, require ceilings below the roofs in order to make a proper finish in the

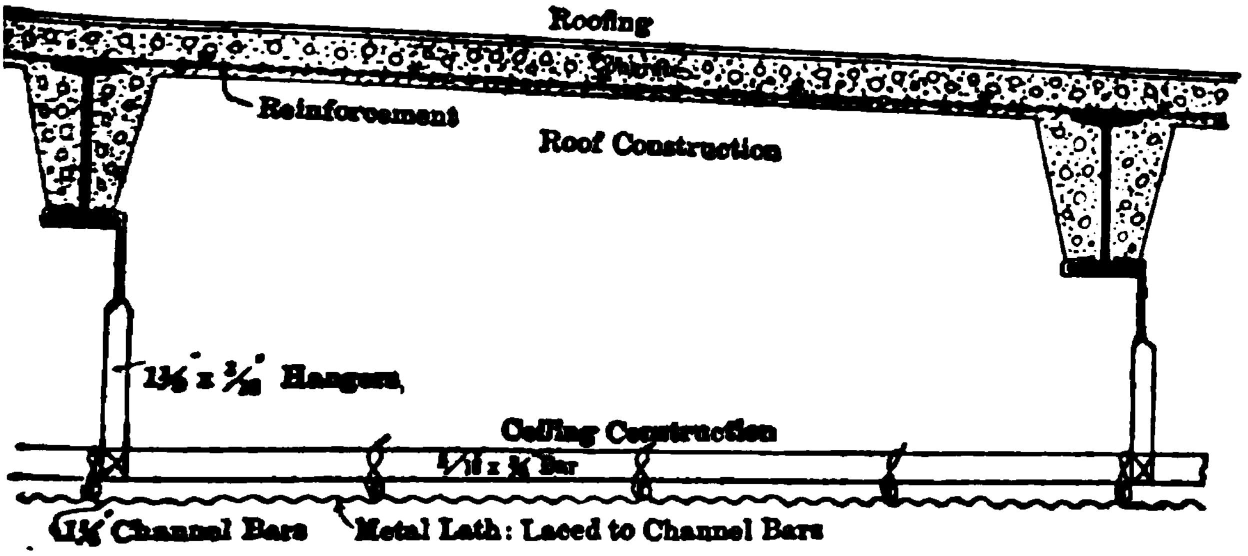


Fig. 57. Suspended-ceiling Construction

...s, and also for heat-insulation. In office-buildings the ceilings of the top ... are often framed and constructed like the floors, but with a lighter con...tion. More often the ceilings are suspended from the roof, as this requires ... less steel and is consequently much cheaper. It answers the purpose ... as well, that is, if the roof-beams are efficiently protected. Fig. 57 shows

a common construction for such ceilings. Wrought-iron hangers, about 1¼ 3/16 in or 1 by ¼ in, split at one end to hook over the lower flanges of roof-beams, are used to support 5/16 by ¾-in flat steel bars, spaced about 4 f centers; and to the under-side of these are laced ¾-in, ⅞-in, or 1¼-in chan 12 or 16 in on center receive the metal lat The bottom of each ha is bent at right-angle form a seat for the and the bar is laced to hangers. No bolting riveting is required, all nections being made lacing wire, or by ben the iron. Where stiffe wire lath is used, the c nels may be spaced 1 on centers; but if the dinary expanded laths used, it is better to p

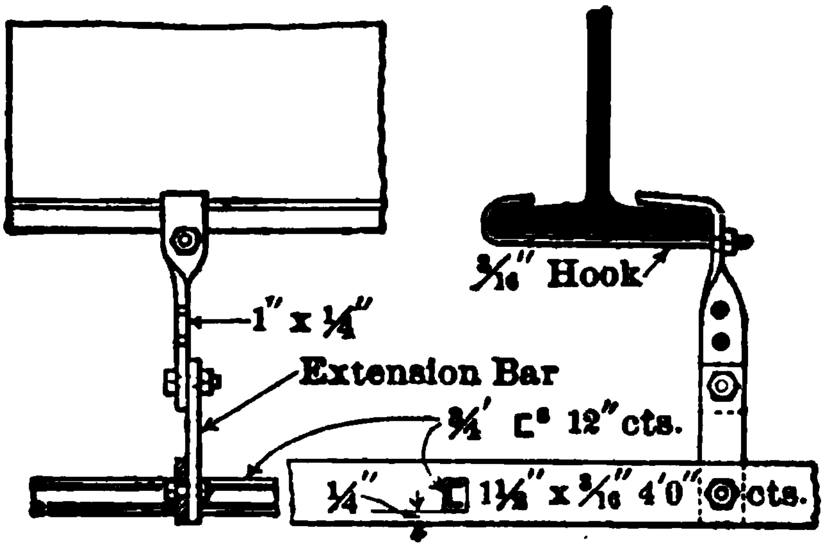


Fig. 58. Suspended Ceiling. Details of Two-bar System

the channels 12 in on centers. If ordinary lime mortar is used for plasterin 12-in spacing is really necessary. Another system is one which uses one set of horizontal bars, which are spaced close enough to receive lathing, and which are supported by hangers. With stiffened wire lathing, roof-beams spaced not over 5 ft apart, and short hangers, this may be the cheaper system; but without the stiffened lathing, there is no stiffness to the ceiling at right-angles to the bars. Where the hangers are 3, 4, or 5 ft long, and the spans between the beams

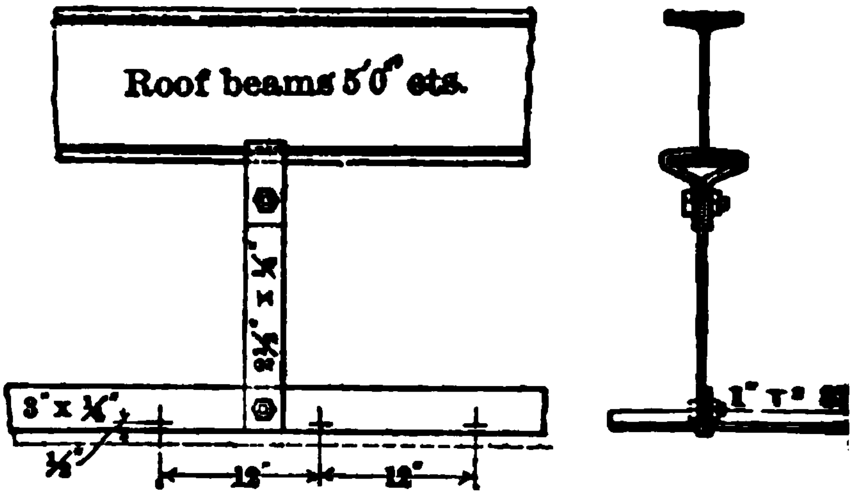


Fig. 59. Suspended Ceiling Details of Two-bar Sys

wider than 5 ft, the two-bar system, shown in Fig. 57, requires less steel, the reason that the channels, having spans of only 4 ft, may be made light, and only one third or one fourth the number of hangers are required. place of the small channels, small T bars or flat bars may be used, but when bars are held by lacing, channels are preferable.

Figs. 58* and 59* show very satisfactory details for the construction of two-bar system. Instead of the hook shown in Fig. 58, the hanger may split at the top, one half bending around one side of the beam-flange and other half around the other side. Where the ceiling is suspended below te cotta arches, toggle-bolts are used for the support of the hangers. The of the small bars supporting the lathing are usually spliced by means of sh iron clamps, about 6 in long, wrapped closely around the bars and hamm tight. For suspended ceilings under segmental or paneled floor-construc the same methods are employed, except that the hangers are replaced by holding the ceiling-bars close to the soffits of the beams.

* From Fire Prevention and Fire Protection, J. K. Freitag, pages 687 and 688.

6. Partitions and Wall-Coverings

equirement of Fire-proof Partitions. As a rule the partitions in fire-of buildings are not required to support any weight, but merely to serve the pose of dividing the spaces into rooms, and to confine a fire to the compart-nt in which it originates. No greater strength, therefore, is required in a tition than is necessary to carry its own weight. Rigidity, however, is re-red, and a rigidity in proportion to its height and unsupported length. When itions separate apartments or sections of a story, that is, when they are tically without window-openings or door-openings, they should be rigid ugh to prevent the passage of water from a hose-stream as well as the pass-of flame. In other cases this may be unnecessary; in fact, at times it may n desirable to construct partitions which can be easily removed to get at e spreading through doors or windows. The materials of partitions should incombustible. They should be poor conductors of heat. It is desirable, , to have them unaffected by water. Lightness is a good property, as any ease in the dead weight of the construction adds to the cost of the structure. titions should be as sound-proof as possible. Window-openings should be ided, when possible, in fire-proof partitions, and even door-openings should reduced in number to a minimum. In many buildings, however, in which s have no openings into streets or courts, such windows are necessary for ting the halls. When this is the case the frames should be made fire-proof, e-glass should be used, and, if possible, the sash made stationary.

Fire Tests on Partitions. In New York City no materials or types of con-ruction are permitted for interior permanent partitions in fire-proof buildings at have not met the required fire tests. The standard test of the American ciety for Testing Materials is based on the New York test.* Briefly, these ts require that the partition shall resist for one hour the destructive action of a ed fire, the heat of which has been gradually increased to 1700° F. during e first half-hour and maintained at that temperature for the balance of the e; and that it shall resist, also, for two and a half minutes at the conclusion of e fire test, the application of a hose-stream at 30 lb pressure.

Types of Partitions. Fire-proof partitions that are in common use may grouped, according to the materials or the method of construction used, as lows:

(1) Brick;
(2) Hollow tile or terra-cotta;
(3) Concrete (stone or cinder);
(4) Gypsum block;
(5) Plaster or concrete, with metal.

The choice of the materials and the type of construction are largely influenced the character of the building and the purposes for which it is used.

Partition-Walls. For bearing-partitions, that is, those which support floor-ms, there are probably no materials more satisfactory than brick and con-te. The latter may be used either in the form of blocks, or may be poured forms. Dense tile, also, is being used with satisfactory results for bearing-ls. Tests show a crushing strength, on net sections, equal to that of brick.

Hollow-Tile or Terra-Cotta Partitions. These are usually built of blocks, er square or brick-shaped, according to the particular product used. The are blocks are usually 12 by 12 in on the face, and the brick-shaped blocks usually 12 in long but vary in height. Both shapes are made in thick-

* See latest Year Book, Am. Soc. for Test. Mats.

nesses varying from 2 to 12 in. The 3-in, 4-in, and 6-in blocks are comm used, the 4-in blocks being the most popular for ordinary work. For more important partitions, such as stair and elevator-enclosures, not narrower than the 6-in blocks with the double row of cells should be u The blocks are commonly set with the voids vertical. Fig. 60 shows typ

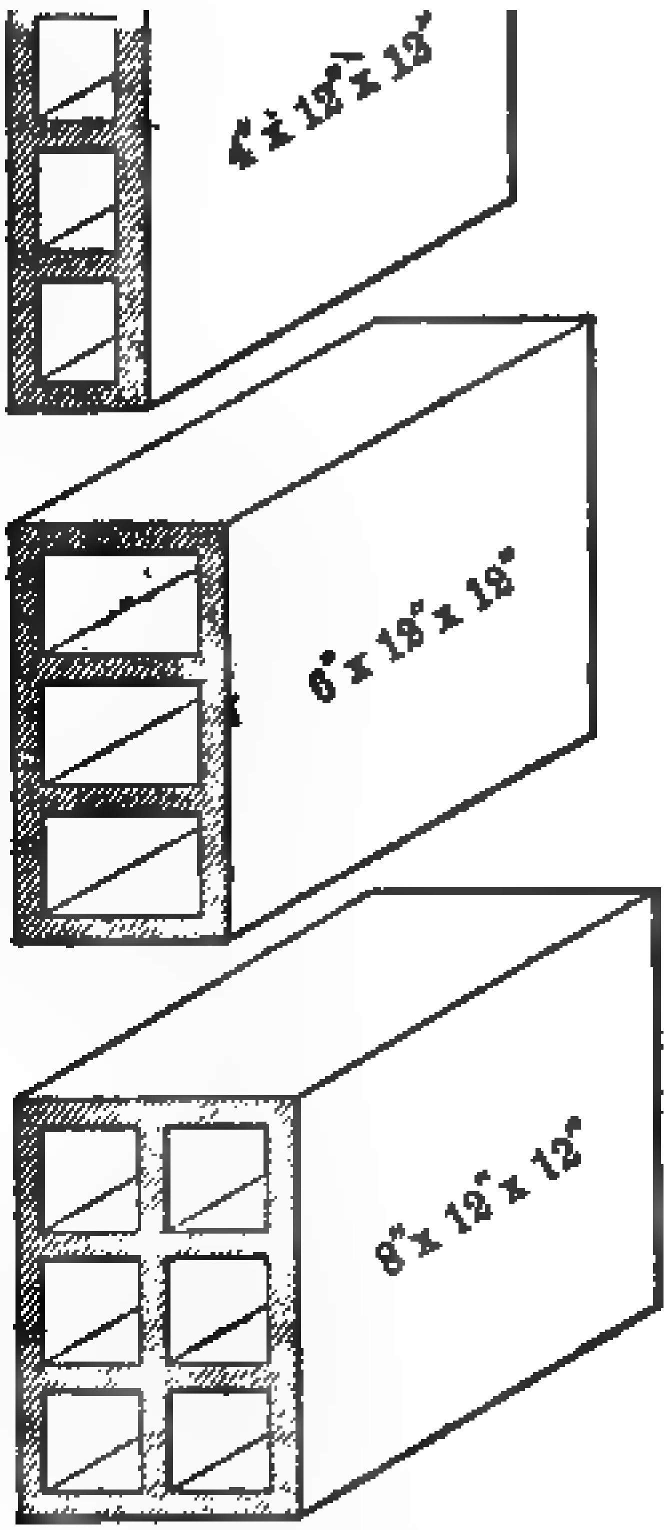


Fig. 60. Hollow-tile or Terra-cotta Partition-blocks

shapes of both the square and brick shaped blocks. Fig 61 shows ro cornered and angle-cornered partition-blocks, which must be set vertic "Terra cotta partitions of a 2-in thickness have been placed on the ma but have not been extensively used. A 2-in terra-cotta partition of any stre or efficiency is quite impracticable, and where floor-area is so valuable more space cannot be occupied, terra-cotta is not the material to be employe

* Freitag.

ugh the addition, however, of band-iron laid between the courses and ated under the name Phoenix,* the strength of a 2-in tile partition is greatly ased. The New York partition, Bevier Patent, consists of 2-in tiles, reind with truss-metal, such as is used in the New York floor-arch. (See Fig. 28.)

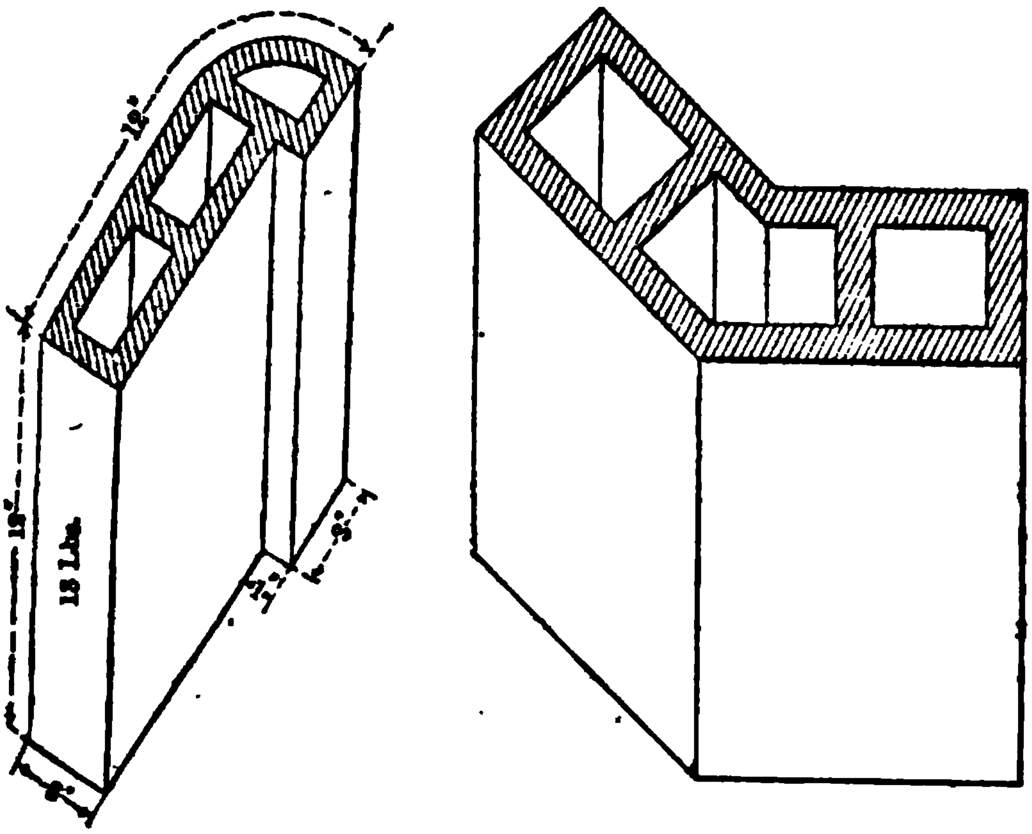


Fig. 61. Hollow-tile Round-corner and Angle Partition-blocks

rous Versus Dense Materials. For inside partitions the POROUS mate- are preferable to the DENSE, while for outside walls the dense materials d be used. With DENSE TILING it is necessary to insert either wooden g-strips, which are very objectionable, or blocks of porous tile to take place. It is becoming daily more difficult to get the sawdust necessary to the porous material.

rtar. Tile partition-blocks should be set in mortar made of one part utty, two parts cement, and from two to three parts sand. The blocks l be well wet before setting and the partition wet down before the plaster- applied.

ights and Lengths of Terra-Cotta Partitions. "The safe HEIGHT ra-cotta partitions in inches may be approximated by multiplying the ess in inches by 40. Common practice allows a safe height of 12 ft for 16 ft for 4-in, and 20 ft for 6-in partitions. For partitions without side- rts the LENGTH should not materially exceed the safe height. Doors and indows may be considered as side supports, provided the studs run from o ceiling."†

ights. The WEIGHTS of either POROUS or DENSE terra-cotta partitions not be taken at less than the following, adopted by the Hollow Building ssociation, as proper average weights:

2-in partition, 14 lb per sq ft;
3-in partition, 16 lb per sq ft;
4-in partition, 18 lb per sq ft;
5-in partition, 20 lb per sq ft;

* Made by Henry Maurer & Son, New York.
† Freitag.

6-in partition, 22 lb per sq ft for one-cell blocks;
6-in partition, 24 lb per sq ft for two-cell blocks;
8-in partition, 30 lb per sq ft;

not including plastering, which adds about 10 lb per sq ft for both sides.

Concrete Partitions. Partitions of stone concrete are seldom used beca of the forms necessary for their erection, which make them comparativ expensive. Unless reinforced they take up too much room. Furtherm they are the heaviest of all partitions. Even in buildings that are entirely reinforced concrete they are not always used. Cinder-concrete partitions somewhat lighter and considerably cheaper than those of stone concrete. even these are too heavy and too troublesome to construct to be satisfacto Among the partitions tested and approved by the New York City Build Bureau is one that consists of cinder-concrete blocks, 2½ and 3 in thick, thicker ones being hollow, 12 in high, and 18 in long. They have their ed cast with tongues and grooves that furnish more or less of a bond between blocks when they are set. Hollow, concrete building-blocks make fairly g partitions, but are objectionable on account of their thickness and weight.

Gypsum-Blocks. The term GYPSUM-BLOCKS is now more generally ployed than the term PLASTER-BLOCKS, as it is more accurately descriptive. principal makes on the ma are the ACME, made by Acme Cement Plaster C pany, St. Louis, Mo.; ANCHOR, made by the Am can Gypsum Company, Clinton, O.; the PYRO made by the United St Gypsum Company, Chic Ill.; the BELL, made by Rock Plaster Manufactu Company; the blocks of Niagara Gypsum Comp and the M. A. Reeb Corp tion, both of Buffalo, N. and the blocks of the Plym Gypsum Company, Dodge, Iowa. The usual of these blocks is 12 in in, although some are 13½ in by 32 in. The thickness is generally 2, 3, 4, 5, 6, and 8 in, for the low blocks, and 2 and 3 in for the solid blocks.

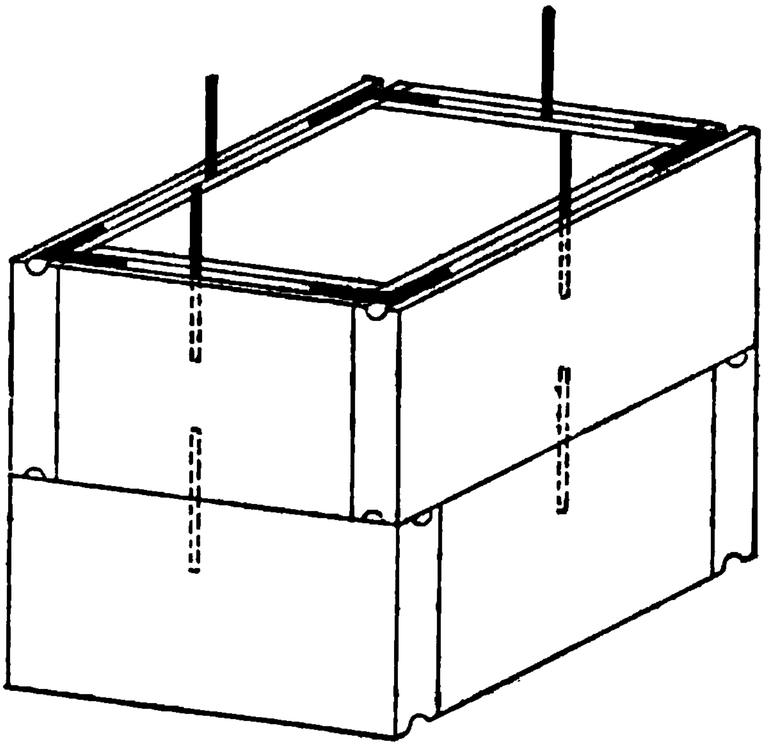


Fig. 62. Plaster-blocks. Doweled Construction.

Gypsum-Block Partitions. Blocks made of gypsum (plaster of P combined with various substances, such as cinders, wood chips, cocoanut asbestos, etc., have been largely used for partitions in fire-proof buildings. principal advantages claimed for these partitions are their great lightnes reduced cost compared with other forms of partitions. Gypsum blocks c readily cut with a saw, and have a considerable holding power for nails. fire tests, made for the Bureau of Buildings, New York City, they have gen shown considerable resistance to the flame and have transmitted less heat partitions of any other form. They did not, however, always stand the stream, some of them being easily pierced, and all of them being more o washed away by the water. An objectionable characteristic of these bloc

ir tendency to absorb moisture while being stored and to draw water from the stering when it is applied. This moisture works down to the bottom the partition where it is likely to injure the wooden base. These par-ns are made in THICKNESSES varying from 2 to 4 in, those less than 3 in hickness generally being solid; and their height should not exceed from 50 50 times the thickness of the blocks, unplastered. Hollow blocks should ays be set with the cells horizontal. The edges of the blocks are generally wed or otherwise arranged so that the mortar joint forms a key between n. In some forms of these partitions the blocks are BONDED together by as of metal dowels,* running across the horizontal and vertical joints from block into the adjoining one, as shown in Fig. 62. The cut illustrates the of the block in the construction of dumb-waiter shafts and shows how the ks are anchored at the corners by iron dowel-angles. Gypsum plaster is l in laying plaster-blocks, and occasionally fibered-gypsum plaster, tempered sand, may be employed. All of the partitions in the newer portions of the adnock Block, Chicago, and in many other prominent buildings of Chicago New York City, are of Gypsum blocks. Gypsum blocks make the lightest tical partition known. The weight of these partitions per square foot may aken as follows;

Thickness of block, inches . .	2	3	4	5	6	8
Weight in lb per sq ft	10	12½	14¼	17¼	19	26

out 8 lb per sq ft should be added to obtain the weight of the partition plastered on both sides.

ackolite. A plaster-block extensively used is the Mackolite Hollow Block, by the A. B. Fireproofing Company, Chicago, Ill. Mackolite partition tiles generally made in the form n in Fig. 63. The 3-in, 3½-in, -in tiles are made 48 and the s 30 in long, all the tiles being high. The blocks are laid in ir courses, breaking joint as t-stone work. Lime mortar ed for setting. In fitting d openings or at angles the are cut with a saw, and this a material saving in time aterial. It is claimed that ocks make very strong par-. The composition of the is plaster of Paris mixed ertain chemicals, reeds, and Reeds of the same length blocks are placed in the molds and the plaster of Paris and fiber are then with water, to which the chemical has been added, and poured around the o that they are nowhere exposed. The reeds give longitudinal strength to cks while the fiber makes them tough and elastic. The material sets in half an hour, after which the blocks are kiln-dried for four days.

FIG. 63. Mackolite Partition-blocks

sinite Partitions. The main feature of these partitions is the GYPSINITE hich is handled and erected in the same manner as a wooden stud in the y non-fire-proof partitions. The stud is composed of wooden nailing- ompletely protected and embedded in a plaster-composition known as

nted by the Sanitary Fireproofing and Contracting Co., New York City.

GYPSINITE CONCRETE. The studs are carefully made and are plumb and t
Metal lath or plaster-boards are secured to the studs and plastered, complet
the partition, which is about 4½ in thick. (Fig. 64.) This partition is sligh
heavier than the ordinary partition of wooden construction. It is q
as stiff and as strong as a good tile or other partition, and the nailing-s
feature of the studding facilitates the application of a wooden trim.
is said to be particularly sound-proof, and the spaces between the st
afford an opportunity to conceal pipes, wires, etc. Gypsinite studs are 3 b
by 12 in, and weigh 3 lb to the foot. They can be made any size requi

Fig. 64. Gypsinite Studs, Metal Lath, and Plaster

In the partitions the studs are usually placed 16 in on centers and bridged
may be required They are fastened to the floor or ceiling by the use of s
and plates of the same material, or by light channel-irons, which are spiked
the fireproofing. The manufacturers believe that in large quantities these st
can be furnished as cheaply as wooden studs and that the partitions can
erected as cheaply as ordinary lath-and-plaster partitions. Gypsinite studs
manufactured by the United States Gypsum Company, Chicago, Ill.

Solid, Plaster-and-Metal Partitions. Thin partitions of plaster appl
to metal lath and metal studs, made solid, and finished about 2 in thick, h
been extensively used in fire-proof buildings. They are remarkably stiff, ow
to the adhesion of the plaster to the steel, and they are lighter and occupy
space than any other practical fire-proof partition of equal strength. In
fire tests these partitions act very much like the plaster-block partitions, re
ing thoroughly the passage of the flames. But the plaster always washes
when the hose is applied and the lath becomes exposed. The rigidity of

al fabric on the metal studding has been considered by firemen a disadvantage, as it is very difficult to cut through it when necessary to get at a fire. construction of these partitions is practically the same for the different ics used, which are described on pages 846 to 850. This lath or fabric aps to be subject to the CORROSIVE EFFECTS of the plaster. In the demolition e Pabst Building, New York City, the metal lath used throughout in the itions was found to be considerably corroded, after about four years, even gh the lath had been painted. On the other hand other cases are cited by

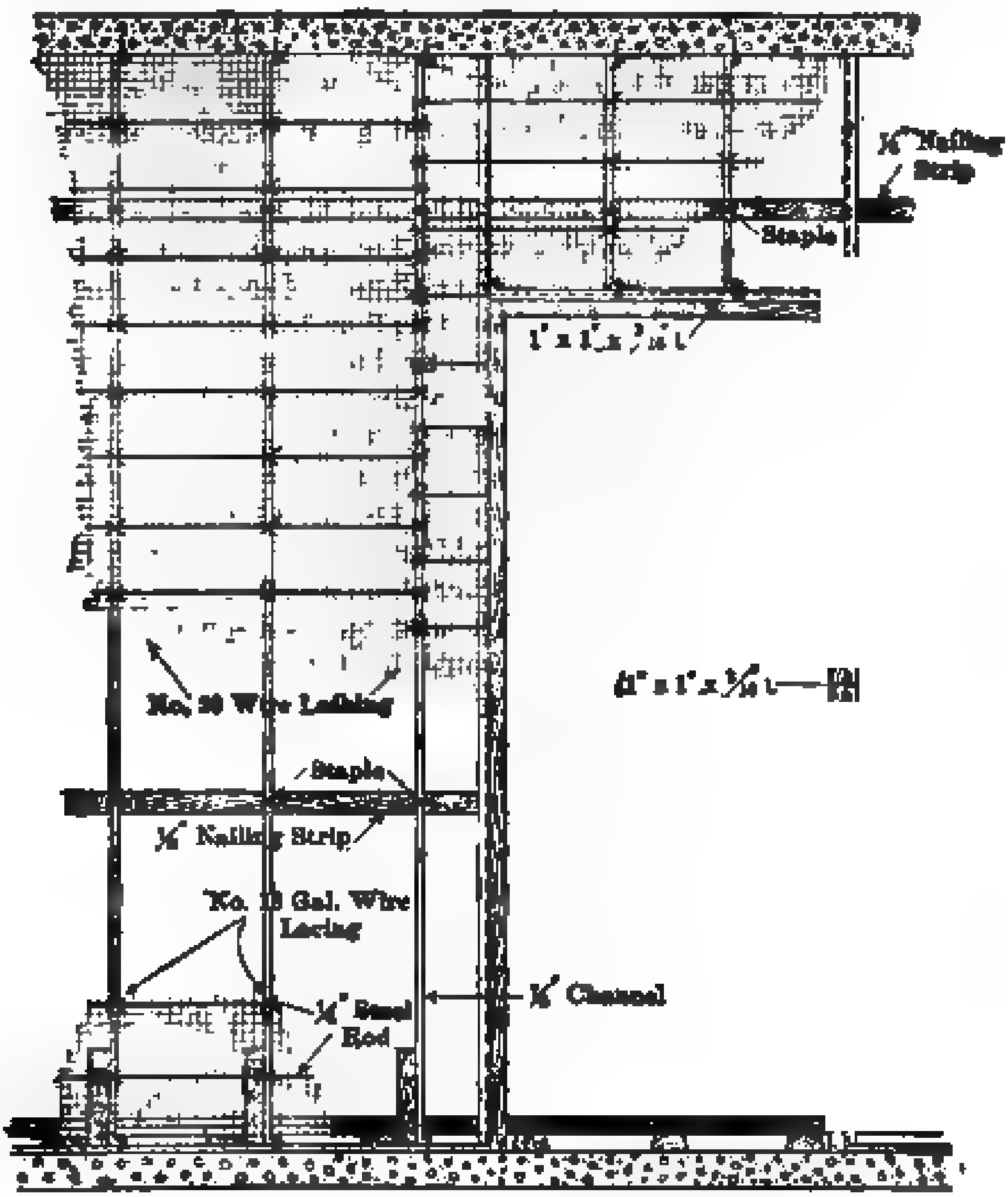


Fig. 66. Two-inch Solid Plaster Partition. Elevation

anufacturers, such as the Chess residence at Pittsburgh, Pa., the Sturtevant nce at Springfield, Mass., and the West End Trust Building at Philadelphia, in which after twenty years no corrosion of the metal lath in plaster parns was observed. The investigations of the United States Bureau of ards of various forms of stucco-construction seem to bear out the manurers' contention. The lath should in all cases be protected against initial ipient corrosion by painting or galvanizing before being embedded in the ntitious material.

ights of Plaster-and-Metal Partitions. The WEIGHT of a 2-in solid ion, when dry, is about 20 lb per sq ft. The weight of partitions of greater

thickness may be estimated on a basis of 120 lb per cu ft for plaster, and for cinder concrete, slightly tamped.

Construction of Solid Two-Inch Partitions. Figs. 65 and 66 show usual method of constructing 2-in partitions. The studs, usually ¾-in o

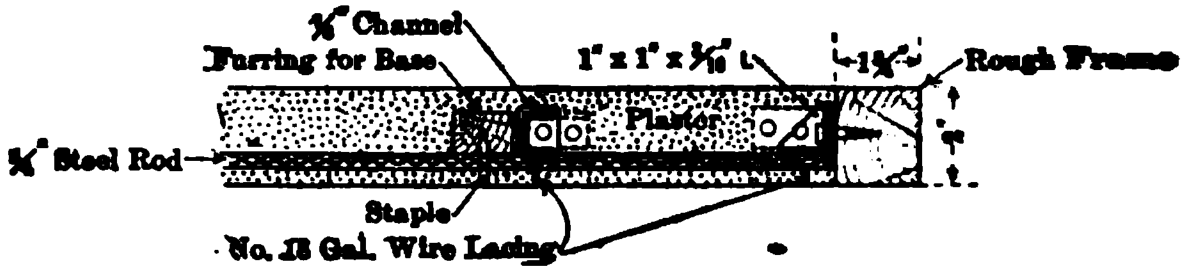


FIG. 66. Two-inch Solid Plaster Partition. Horizontal Section

channels, are bent and punched at the ends, and at the bottom are nail wooden strips, which are first secured to the floor-panels, or to the top steel beams where the partitions come over them. These wooden strips been found necessary as a sort of cushion to allow the studding to expa case of fire. At the top, the studs are nailed to the underside of the floor-p or, if there is a suspended ceiling, they are wired to the bars supportin ceiling. At the openings, 1 by 1 by 3⁄16-in angles are used, and these are every 16 in for No. 12 screws, used in attaching the rough wooden fram the angles. After the studding is in position, the metal lathing is laced t side of it with No. 18 galvanized wire. After the lathing is in place th penter should attach wooden grounds to secure the base, and pegs or spot-gr for chair-rail, picture-molding, etc. These grounds are secured by st and when the partition is plastered, become very rigid. In plastering partitions, five coats of plaster are required to make a good job; a scratcl on one side, a brown coat on each side, and the usual white coat on each si finishing. It is essential for all thin partitions that a HARD-SETTING mort used, such as Acme Cement, King's Windsor Cement, Adamant, or Rock Plaster.* The partitions acquire their STIFFNESS largely from the solidity plastering, hence the firmer and harder the plastering the more substanti walls.

Double Partitions. Electric wires and ½-in gas-pipes can be run in th SOLID PARTITIONS; but if it is desired to run larger pipes, DOUBLE PARTI

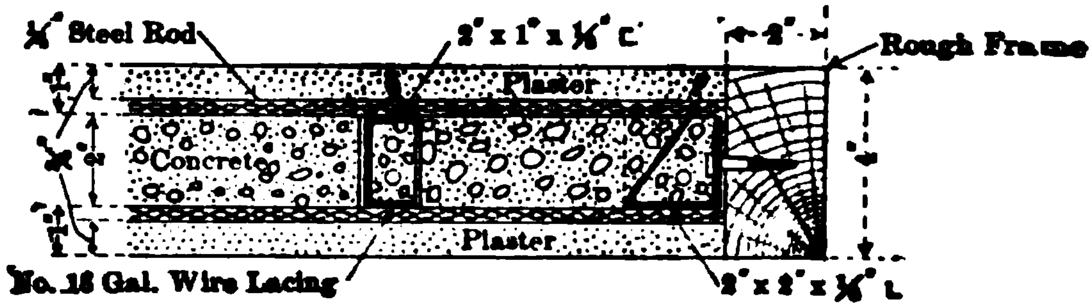


FIG. 67. Four-inch Solid Plaster and Concrete Partition. Horizontal Secti

that is, partitions with lathing on each side of the studding, must be used these partitions, 2-in, 3-in, or 4-in channels, or flat bars set edgewise, may b sheet-steel channels being probably the most economical. When the between the studs is not filled with mortar or concrete, the DOUBLE PAR

* Made respectively by the Acme Cement Plaster Company, St. Louis; J. B King & Company, New York; the United States Gypsum Company, Chicag and the Rock Plaster Manufacturing Company, New York.

at stand fire and water as well as the SOLID PARTITION, while it is much more sive.

astruction of Solid Four-Inch Partitions. Fig. 67 shows a partial sec- hrough a SOLID PARTITION finishing 4 in thick when plastered. It has great gth and resistance to fire and water, and affords nient spaces for pipes and thicker jambs for frames. These partitions have cores of cinder ete, with metal lath on both sides, and are plas- in the usual way. As the concrete will receive no wooden furring is required to fasten the base- s, chair-rails, and picture-moldings in place.

rger's Economy Studding and Furring. Fig. lustrates a patent stud manufactured by the r Manufacturing Company, Canton, Ohio. It de of No. 18 or No. 20 sheet steel, and in five varying from ¾ to 1¼ in. The peculiar ad- ge of this stud is the provision for attaching the For this purpose prongs are punched from sides of the flange, which are left standing at angles to the face of the flange. The lath is l against the stud, the prongs pressed through eshes and then turned up over the lath with a er. This fastens the lath more firmly and ly than by any other method. The ends of the are secured by sockets which are fastened to the and ceiling, a clear space being left above the f the studs for expansion. Where partitions ect or where there are angles, angle-irons with s are used in place of the T irons. By using studs and expanded-metal lathing, a saving in an be effected over the construction shown in 6. These T's are used, also, for supporting nded ceilings under I beams, the T's being d to the flanges of the beams by specially de- clips. Furring-strips and channels, also, are on the same principle.

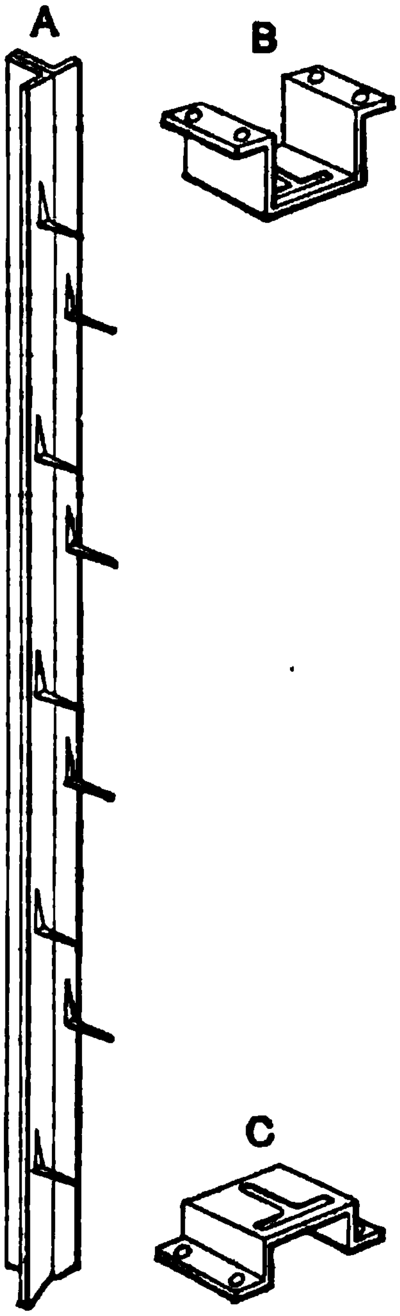


FIG. 68. Berger Studding or Furring and Stud-sockets

cing of Studs in Two-Inch Solid Partitions. in solid partitions with ¾-in rolled channels or conomy Studs, the studs should be placed 12 enters when the height of the story exceeds 10 ft. When the height of ry is less than 10 ft, a spacing of 16 in will answer. For hollow partitions -in studs, the studs can be spaced 16 in on centers for story-heights of 61 less. For greater heights they should be placed 12 in on centers.

Stud. In Fig. 69 is shown the Rib Stud made by the Truscon Steel ny, Youngstown, Ohio. It is made in widths of 2¼, 3¼, 4¼, 6¼, and and in lengths up to 18 ft. The studs are made of OPEN-HEARTH STEEL, o-rib studs weighing 0.55 lb per ft and the three-rib studs, 0.85 lb per ft. n SOLID PARTITIONS with ¾ or 1-in channels or studs, the studs should ed from 12 to 16 in on centers, depending upon the stiffness and rigidity ath. A 12-in spacing should never be exceeded when the height of story than 12 ft. For HOLLOW PARTITIONS with 2-in studs, the studs can ed 16 in on centers for story-heights of 16 ft or less, when No. 24 (United gauge) expanded metal or No. 18 (United States gauge) wire lath, 2½ by

2¼ mesh, are employed. For greater heights the spacing should never ex
12 in. No. 22 (United States gauge) expanded metal, weighing at least 4
per yard, and No. 20-gauge V-stiffened wire lath or wire lath with rods or
eners spaced 7¼ to 8 in on centers, give satisfactory rigidity for both parti

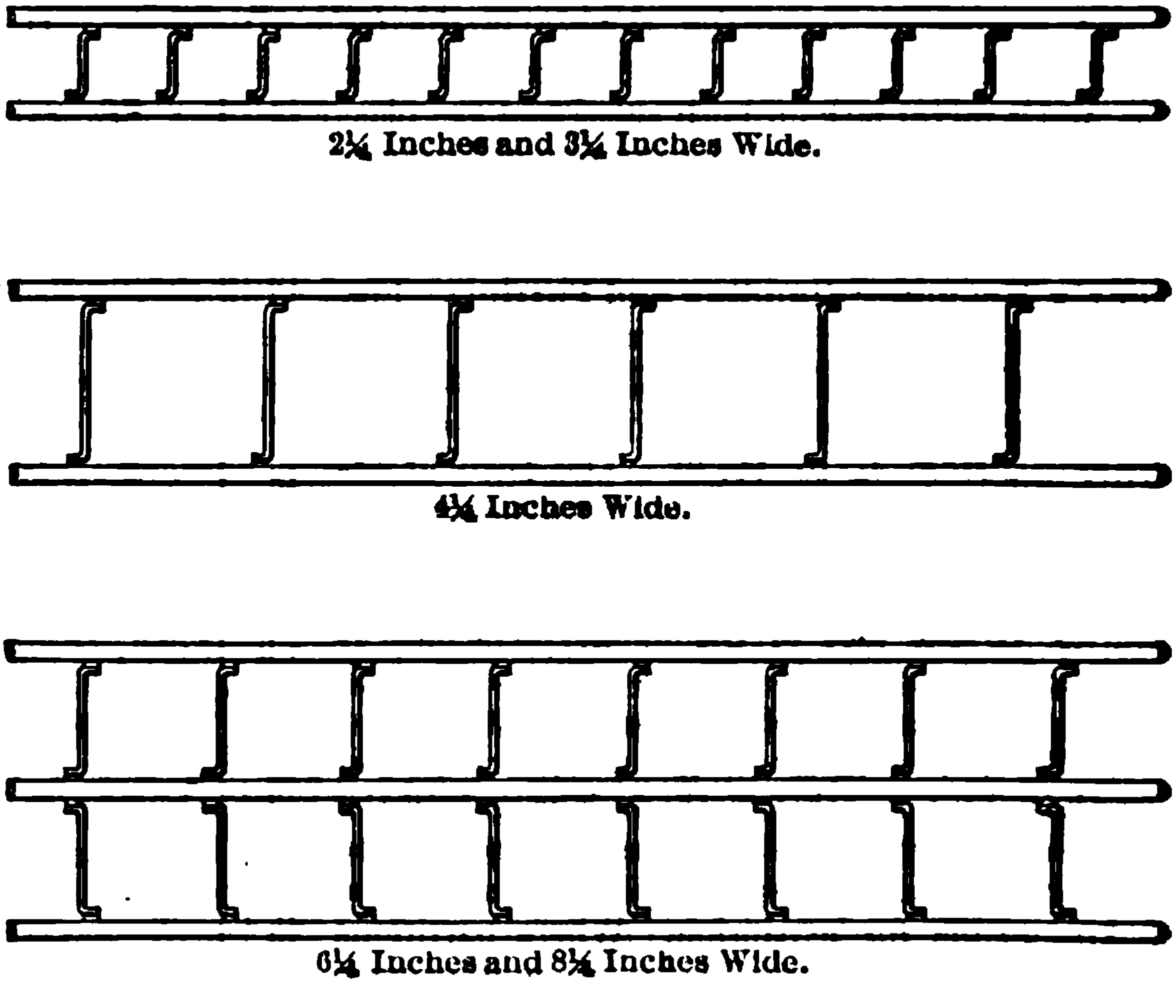


FIG. 69. Rib Stud for Plaster Partitions

and ceilings when the studs or furring-strips are set 16 in on centers. I
should be wired to the metal studding with No. 18-gauge annealed galvan
wire.

Metal Lath. Numerous styles of METAL LATH have been put on the ma
in recent years to provide for a cheap, light, and thin partition-construct
For fire-proof buildings ME
STUDDING should always
used. Metal lath is supp
either plain, painted, or
vanized. It is recommen
that metal lath be alway
least painted, to prevent ir
corrosion until the lath ca
covered by the mortar.
vanizing is necessary w
there is danger of mois
reaching the lath while
without a protective coa
lime or cement. Where a
ticular type of lath is

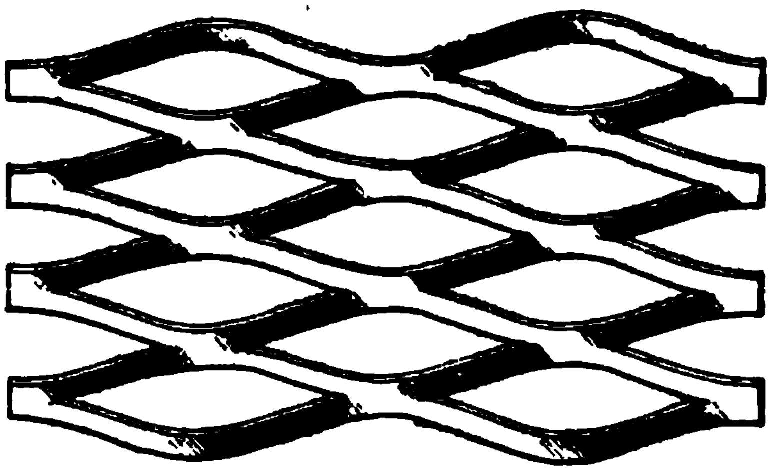


FIG. 70. Expanded-metal Lath with Diamond-shaped Mesh

mentioned in a specification, it should be generally described as foll
"Painted or galvanized No. 24-gauge expanded-metal lath, weighing not
than 3½ lb per sq yd, or painted or galvanized woven-wire cloth, No

ge, 2½ meshes to the inch, with stiffeners placed 8 in on centers and weigh- not less than 3⅓ lb per sq yd." Metal lath should be so made that it will plaster freely, key it thoroughly, and wholly embed itself in it. These are acteristics of expanded-metal and woven-wire laths which make them rior to sheet lath. Sheet laths are economical in the use of mortar, which ly covers one side of the lath and latches through the perforations without oughly embedding the metal. The difficulty of stretching plain wire lath enough to make a firm foundation for plaster and the resulting necessity close spacing of the studs to secure the required bearing, has led to the duction of stiffened wire cloth and ribbed or corrugated expanded metal der to obtain the necessary rigidity. To overcome the necessity for separate ing-studs, expanded-metal and sheet-metal laths are manufactured also in a piece steel lath-and-stud.

PANDED METALS differ in the process of manufacture. One type is made mply slitting the sheet and deploying it into the diamond shape; the other is made from thin s of soft, tough by a mechanical ss which pushes and expands the l into the mesh at the same time ses the direction e edge, so that the urface of the cut d is nearly at angles to the al surface of the . It is claimed the COLD WORK- of this low-carbon steel increases the ELASTIC LIMIT and ULTIMATE NGTH. In specifying expanded metal, it is necessary to give the weight of nished product per square yard as well as the gauge of metal, as the strands be of various widths. EXPANDED METAL is made either with diamond-shaped 70) or rectangular (Fig. 71) meshes. When laid with the long strands per- icular to the studs, the lath with the rectangular mesh is the stronger of the Rigidity is also obtained by corrugating and expanding the metal in various , which make the so-called ribbed, corrugated, and integral laths. WIRE l is stiffened by clipping corrugated-steel furring-strips to the lath or by ing or welding rods or V-shaped stiffeners at regular intervals.

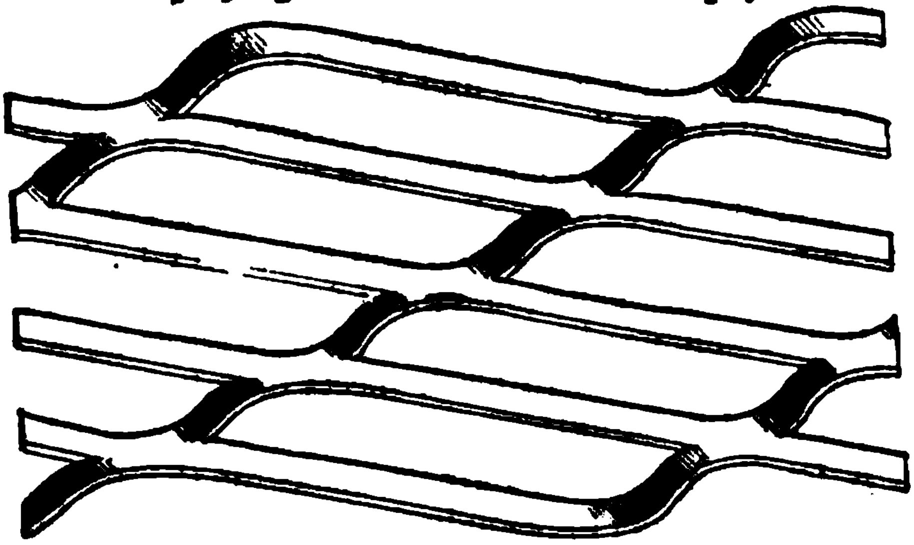


FIG. 71. Expanded-metal Lath with Rectangular Mesh

pes of Metal Lath. Metal lath may be classified as follows:

Expanded-metal lath;
- (a) Diamond and rectangular mesh,
- (b) Ribbed or corrugated,
- (c) Integral, combining functions of both lath and studding,

Sheet lath;
- (a) Flat perforated,
- (b) Integral, combining functions of both lath and studding,

Woven-wire lath;
- (a) Plain,
- (b) Stiffened.

e of the laths and their characteristics are given in the following para- s.

(1) Expanded-Metal Lath

Rotary Diamond-Mesh Lath. This lath is made by the Berger M facturing Company, Canton, Ohio. It is furnished in sheets 18 by 96 i Nos. 27, 26, 25, and 24 gauge, weighing respectively 2⅓ lb, 2½ lb, 3 lb, an lb per sq yd. It is made of Toncan Metal, for which greater HOMOGENEI claimed than for charcoal-iron and steel, and less liability to CORROSIO PITTING.

Bostwick Lath. Bostwick lath is made by the Bostwick Steel Lath C pany, Niles, Ohio. It is furnished in sheets 14 by 96 in, approximately 1 sq and also 24 by 26, and is made in Nos. 24 and 27 gauge.

Steelcrete Lath. This material is manufactured by the Consolid Expanded Metal Companies, Braddock, Pa., and is furnished in two st known as STEELCRETE A lath and STEELCRETE B lath, for exterior stucco-w and in the standard-form DIAMOND LATH, extensively used for exterior an terior plastering-work. STEELCRETE DIAMOND LATH is divided into three d nations, P LATH, F LATH, and H LATH. The P LATH meets the specificatio the United States Post Office Department, weighs 4.37 lb per sq yd, and is m factured from 22-gauge material in sheets 24 by 97 in. The F LATH is manu tured in sheets 24 by 97 in from the gauges 24, 25, 26, and 27, respecti weighing 3.40, 3.00, 2.55, and 2.33 lb per sq yd. The H LATH has a size of s 28 by 97 in and is manufactured from the gauges 24 and 26, weighing resp ively 2.90 and 2.20 lb per sq yd. STEELCRETE LATH can be obtained manu tured from open-hearth black sheets or galvanized sheets, or in copper-bea sheets (acid-resisting).

A Diamond-Mesh Lath is made by the Penn Metal Company, Bo Mass., in sheets 24 by 96 in in size and of the following gauges: No. 22, weig 4 lb per sq yd; No. 24, weighing 3.4 and 3 lb per sq yd; No. 26, weighing 2 per sq yd; and No. 27, weighing 2.3 lb per sq yd. For such extraordi conditions as are found in gas-plants, dye-works or places where exces moisture or salt-air action exists, Hampton Rust-Resisting Lath is made.

Key Expanded-Metal Lath, made by the General Fireproofing Comp Youngstown, Ohio, is furnished in sheets 24 by 96 in, in Nos. 27, 26, 25, an gauge, weighing respectively 2.34, 2.50, 3.00, and 3.40 lb per sq yd.

Kno-Burn Lath, made by the North Western Expanded Metal Comp Chicago, Ill., is furnished in sheets 18 by 96 in, in Nos. 27, 26, 25, and 24 ga weighing respectively 2⅓, 2½, 3, and 3.4 lb per sq yd. When made fro special ACID-RESISTING sheet steel, this lath is sold under the name XX Cen Expanded Metal Lath.

Herringbone Expanded Metal Lath (Fig. 72), made by The General proofing Company, Youngstown, Ohio, is furnished in three styles, A, AAA, BB. STYLE A is made in sheets 13½ by 96 in (1 sq yd), of No. 28-gauge m weighing 3 lb per sq yd. STYLE AAA is made in sheets 18 by 96 in, and from 26, and 24-gauge metal, weighing 2.53, 2.81, and 3.79 lb per yd, respectiv STYLE BB is made in sheets 20¼ by 96 in (1½ sq yd), of Nos. 27, 26, and 24-g metal, weighing respectively 2¼, 2½, and 3¾ lb per sq yd. It is made of s American ingot-iron, or galvanized sheets. Ribs are set across studs and d down towards them.

Sykes Expanded Cup-Lath, made by the Sykes Metal Lath and Roo Company, Niles, Ohio, is furnished in sheets 18 by 96 in, with an anti coating, or painted black, or galvanized. It is made of Nos. 27, 26, and gauge metal, weighing respectively 2.8, 3, and 3.7 lb per sq yd.

andard Rib Lath, made by the Truscon Steel Company, Detroit, Mich., rnished in sheets 20¼ by 96 in, in grades 1, 2, and 3, weighing respectively ,3.42, and 4.10 lb per sq yd. This company makes also the BEADED PLATE

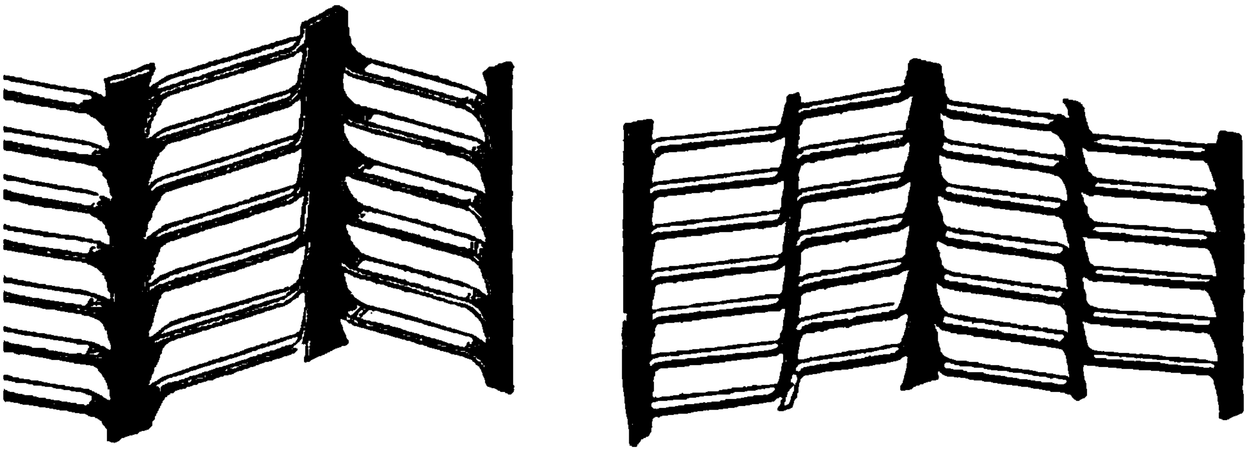


FIG. 72. Expanded-metal Lath, Herringbone Mesh

ATH, which is about 35% heavier and more rigid, permitting wider spacing : studs.

tmesh Diamond Expanded-Metal Lath is manufactured by the Mil-e Corrugating Company, Milwaukee, Wis. This lath is furnished in , 24 by 96; in 27, 26, 25, and 24 gauges, painted; and in 26 and 24 gauges hot-galvanized after cutting. This Company, also, makes CORRUGATED LF-FURRING LATH, in sheets 21½ by 96 in, same gauges, except No. 25, as TMESH.

o-Fur Lath, made by the North Western Expanded Metal Company, go, Ill., is furnished in sheets 22 by 96 in, of Nos. 24, 25, 26, and 27-gauge , weighing respectively 3.80, 3.36, 2.82, and 2.62 lb per sq yd. This lath ls running obliquely across the sheets at the same angle as the strands mesh. This corrugation is said to give the lath greater RIGIDITY so that be used on 32-in centers for walls and on 24-in centers for ceilings. The ations act as furring-strips. It is made from a special ACID-RESISTING and is always supplied painted.

egral Expanded-Metal Lath. Truss-metal lath, Fig. 73, made by the can Rolling Mill Company, Middletown, Ohio, is furnished in sheets

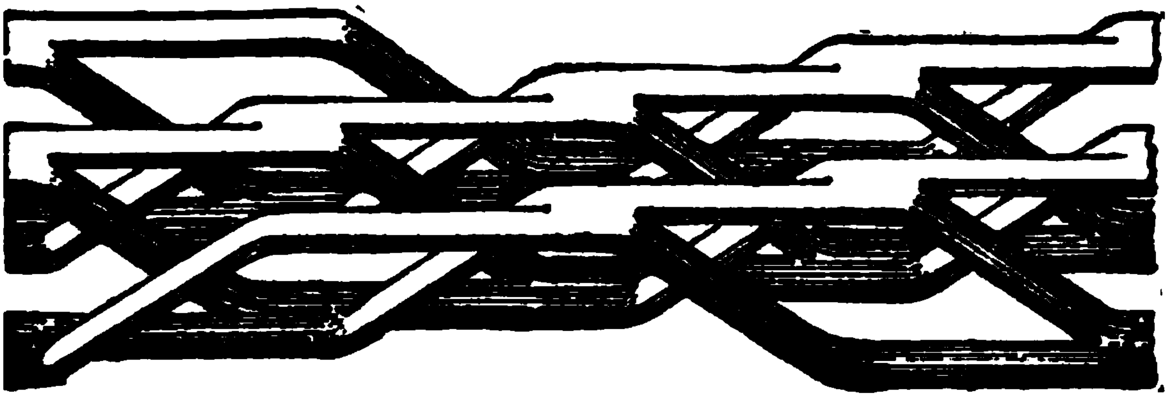


FIG. 73. Truss Metal Lath

)0 in, of Nos. 26 and 28-gauge metal, weighing respectively 80 and 66.7 lb) sq ft. A partition constructed of this lath in one of the test-structures mbia University, New York City, passed through and withstood, with-y sign of distress, the fire and hose-streams of five successive tests.

-Sentering, made by the General Fireproofing Company, Youngstown, s furnished in sheets 29 in wide and in lengths varying by 1 ft, from 4 to

12 ft, of Nos. 28, 26, and 24-gauge metal, weighing respectively 0.58, 0.70, 0.93 lb per sq ft. The width of 29 in is the covering capacity, as laps are vided for by outside ribs. (See, also, page 853.)

Hy-Rib, made by the Truscon Steel Company, Detroit, Mich., is furnish three types known as 4-Rib, 3-Rib, and Deep Rib. The first is in sheets in wide, and the others in sheets 14 in wide. (See, also, page 853.) All style furnished in Nos. 24, 26, and 28-gauge metal. The standard lengths are 6, and 12 ft. Other lengths below 12 ft are cut, but the waste is at the cost o purchaser. Hy-Rib sheets interlock at the sides and ends. In ordering allowance need be made for side laps, but for end-laps 2 in should be allowe laps over supports, or 8 in between supports.

Trussit is manufactured by The General Fireproofing Company, You town, Ohio, in sheets 19 in in width, and in lengths of 8, 10, and 12 ft, from 26, and 24 gauge, weighing 0.57, 0.62, and 0.83 lb per sq ft, respectively.

(2) Sheet Lath

Clinched Lath, made by the American Rolling Mill Company, Mi town, Ohio, is furnished in sheets 13½ by 96 in (1 sq yd), of No. 30-gauge m weighing 4½ lb per sq yd.

Truss-Loop Lath, Fig. 74, made by the Bostwick Steel Lath Comp Niles, Ohio, is furnished in sheets 13½ by 96, 16¾ by 80, 24 by 96, and 2 96 in, weighing 4½ lb per sq This lath is furnished painted u otherwise specified.

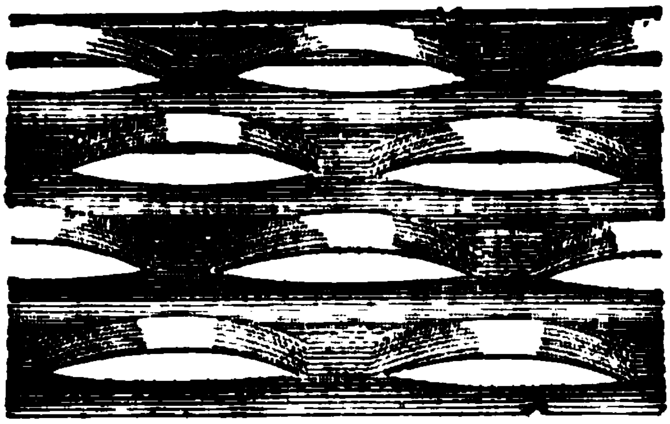


Fig. 74. Bostwick Truss-loop Lath

Genfire Sheet-Steel L made by the General Fireproo Company, Youngstown, Ohio, furnished in sheets 24 by 96 weighing 4.6 lb per sq yd, pai unless otherwise specified.

Sykes Trough Sheet L made by the Sykes Metal L and Roofing Company, Niles, O is furnished in sheets 13½ by 9 (1 sq yd), 15½ by 96, 18½ by 96, and 23¾ by 96 in, weighing 5 lb per sq and made with an antirust coating, or painted or galvanized.

Integral Sheet Lath. Rib-Truss, made by the Berger Manufactu Company, Canton, Ohio, is furnished in widths of 24 in, and in stock length 4, 5, 6, 8, 10, and 12 ft, as follows:

Gauge	Weight per square yard in pounds		
	1½-in rib	¾-in rib	1-in rib
27	73	78	83
28	81	86	92
26	88	94	100
24	117	125	133

(3) Woven-Wire Lath

Woven-Wire Lath is furnished with or without STIFFENERS, which are either h or V-SHAPED RIBS running through the wire mesh to reinforce and stiffen it. is supplied painted or unpainted, or it is galvanized after weaving. It can be nished to order in any required width up to 10 ft. In widths less than 18 in, re is a small charge for STRIPPING. Before ordering, it is very important to ertain the proper width, especially of stiffened lath, as it is desirable to have edges of the lath lap at the supports where it is laced to iron furring. When lath is not of the proper width the results are not so good and there is liable e a waste of material. The standard width of PLAIN and of V-RIB STIFFENED h is 36 in. When beams or studs are spaced 16 in from center to center, the h should be 32 or 48 in wide.

The Clinton Stiffened Lath has corrugated-steel FURRING-STRIPS attached ry 8 in, crosswise of the fabric, by means of METAL CLIPS. These strips con- ute the FURRING, and the lath is applied directly to the underside of the floor- ts, or to planking, furring, brick walls, etc. This lath is made in 36-in widths, h 2½ meshes to the inch, and comes in 100-ft rolls. The manufacturers of lath make, also, a lath STIFFENED WITH ROUND RODS, ⅛ to 1 in in diameter, ced from 8 to 12 in apart. It can be had either galvanized or japanned, and hicknesses from 18 to 21 gauge. Clinton plain WIRE LATH is furnished in rolls ft long.

The Roebling Standard Wire Lath (controlled by the New Jersey Wire th Company, New York), is made of plain WIRE CLOTH, in which, at intervals ½ in, STIFFENING RIBS are woven. These have a V-shaped section and are made of 24 sheet steel, ⅜ and 1 in in depth The n rib is the standard size for lathing on dwork. This lathing requires no furring, is applied directly to woodwork or to s with steel nails driven through the bot- of the V, as shown in Fig. 75. The No. V-rib stiffened lathing affords a satisfac- surface for plastering, when attached to s or beams spaced 16 in apart. The 1-in b lathing is used for furring exterior s. It provides an air-space between the and plaster. Where this lath is to be ied to light iron furring, a 3/16 or ¼-in steel rod is substituted for the V-rib, the lathing is attached to light iron ng with lacing wire. This lath is distinguished from the others by the term -Rib Stiffened Wire Lath. The Roebling lath, whether plain or stiffened, de with 2 by 2, 2½ by 2½, and 2½ by 4-in mesh, the last named being n as CLOSE WARP. The 2½ by 2½ mesh is adapted to all plasters contain- he usual proportion of hair or fiber. The 2½ by 4-in mesh should be used ard plasters and thin partitions. The lath can be furnished in widths up to the rolls averaging 50 yd in length.

FIG. 75. Roebling V-rib Stiffened Wire Lath

all-Boards. There are various forms of WALL-BOARDS of an incombustible e, most of them made of gypsum in combination with felts. They can be as substitutes for laths. They are very light and require but little plaster- aterial. When this saving is taken into account, wall boards cost less than lath and but little more than wooden lath with three coats of plaster. The

boards are generally 32 by 36 in in size, and 1/4, 5/16, 3/8, or 1/2 in in thick
The thinnest boards (1/4 in) weigh 1 1/2 lb per sq ft, and the thickest (1/2 in) 2
Wall-boards of asbestos are described on page 819. The best known is TRAN made by the H. W. Johns-Manville Company, New York.

Sackett's Wall-Board. This is a composite board of three layers of gypsum and four thin layers of wool-felt. The boards can be nailed to woo studding or set flat against solid beams or planks, and can be cut with a s For plastering, the best results are obtained by applying first a brown coa hard wall-plaster, 1/4 to 3/8 in thick, and when this is thoroughly set, finishi with a thin coat of regular hard finish of lime-putty and plaster. Tests investigations at the Underwriters' Laboratories "have shown Sackett Bo Perfection Brand, to be suitable as a base for fibered-gypsum plasters; and w attached to walls, ceilings, and partitions and coated with 1/2 in of plaster, pos fire-retarding properties considerably higher than those of wooden lath w gypsum or lime-and-cement plaster." The Perfection Brand, Sackett's W Board, is 3/8 in thick, and is attached with No. 10 1/2, 1/4-in, flat-headed, 3/4-bar wire nails, 1 1/4 in long, and spaced not more than 6 in at each support. SACKE BOARD is made by the United States Gypsum Company, Chicago, Ill., and Grand Rapids Plaster Company, Grand Rapids, Mich. Other makers of gyp wall-boards are the J. B. King Company, New York (Diamond Brand), Southern Gypsum Company, North Holston, Va. (Economy Brand), and American Gypsum Company, Port Clinton, Ohio (Monarch Brand).

Metal-Rib Plaster-Board is composed of alternate layers of strong absorb paper reinforced with fine annealed wire about 2 in on centers, and stiffe transversely with 1/2-in iron bands, No. 32 gauge, placed 8 in on centers. material is made up to a total thickness of about 3/16 in, impregnated wit coal-tar product, and provided every 2 in with 3/16-in circular holes to key plaster. This is added to the adhesive effect of the absorbent paper. I furnished in rolls 85 ft long and 34 in wide, nailed directly to the studs or be set 12 or 16 in on centers, and lapped 2 in at all joints. This board is rec mended for use with hard-plaster mortars only, and forms a satisfactory b for three-coat work, in which the lap-joint obviates the cracking frequen associated with ordinary plaster-board construction.

Bestwall. BESTWALL is primarily intended for use as an interior finish side walls and ceilings in buildings of all classes. It may be used in all situati where finishes of lath and plaster may be used, and in many situations w the latter finish is not adaptable. It consists of a single layer of fiber calci gypsum, surfaced on each side with specially prepared water-proofed paper curely bonded to the surface. Bestwall is 3/8 in in thickness, and is furnishe stock sizes 47 1/4 in wide, and in lengths of 5, 6, 7, 8, 9, and 10 ft. The finis product presents a smooth, true surface, which is light cream in color on face-side and gray on the reverse side. The edges are slightly beveled to vide for the filling of the joints, and are doubly reinforced. Its weight is proximately 1 850 lb per 1,000 sq ft. Interior finish composed of Bestwa applied by nailing the Bestwall directly to the joists, studs, and furring, filling the joints between the pieces of Bestwall with a specially prepared of the same composition as the core of the board. For the nailing, threepe fine wire nails, spaced from 2 to 3 in at the edges and from 8 to 12 in at the in mediate supports, are used. The filling consists of two operations; first, ROU ING IN and then, TROWELING OUT, to a smooth, true finish, flush with the surf Bestwall is cut and fitted either with a saw or by scoring and breaking ov straight-edge. The completed finish presents a smooth, true, continuous surf

thout showing joints or nail-heads, and ready to receive, if desired, a decoration paint, paper, tint, etc.

Shaft-Construction. The most important partitions in a building are those losing interior shafts. Vertical openings through buildings form flues and me up-drafts. In all buildings, fire-proof as well as non-fire-proof, therefore, y should be inclosed for two reasons: first, to prevent a fire that would find atural outlet in such openings from spreading to other floors; and secondly, prevent, as far as possible, a fire from getting into these openings where the ft would greatly increase its fury. To be thoroughly effective the inclosed ls should be constructed of the same materials as the outside walls of the ldings, namely, brick, stone, or concrete. While they need not be of the ne thickness as the outside walls, 12 in is recommended as a minimum thick-s. In less important structures terra-cotta partitions are sometimes used such inclosing walls. In the walls inclosing elevator-shafts no openings ept those necessary for entrance-doors should be permitted. The doors uld be of fire-proof construction, pages 901–2, and made solid. Glass lights sometimes provided in such doors, although this is not good practice; if y are used, wire-glass, only, should be used, in accordance with the limita-s noted on pages 902–3. Open grille-work for passenger-elevator enclosures eing rapidly superseded by construction which is more fire-resisting. The hitectural features of open grilles may still be retained for the fronts and doors uch elevators by using them in conjunction with approved wire-glass con-ction. In interior light-shafts and vent-shafts, openings must necessarily be vided, but here again the construction of the window-frames, sashes, and ing should be as far as possible as described on pages 901 to 903. When-the occupancy of a building admits, the stairs, also, should be inclosed in onry walls, with fire-proof doors at the openings. Unless so inclosed the rways form flues for the flames, and the stairs themselves, consequently, exposed to intense heat. In such situations, even absolutely fire-proof stairs d not be used during a fire, and possibly it is for this reason that greater s have not been taken to make them fire-proof. Shaft-walls should in all s be carried 3 ft or more above the roof.

eadening Properties of Partitions. The resistance to the passage of d through fire-proof partitions is an important consideration in buildings for living-apartments; and where the rooms are to be used as music-studios, comes a matter of still greater importance. In January, 1895, some tests made to determine the RELATIVE DEADENING PROPERTIES of the different itions shown in Fig. 76, the object being to decide upon the construction should be used in Steinway Hall, Chicago, Ill. The rank of the different itions tested, IN SOUND-PROOF EFFICIENCY, is shown by the numbers at the of the partition-diagrams. The 4-in porous partition was used, but was a success. In the Fine Arts Building, in the same city, double partitions, ar to No. 1, were used, and it is said that they were a great success. It is rising to note that in the tests above mentioned, the 2-in-solid-plaster par-n, No. 3, plastered with common mortar, ranked higher in SOUND-DEADENING ERTIES than those with double studs. In 1892 C. L. Norton tested the D-DEADENING PROPERTIES of partitions of several forms, for the purpose of ting a construction which was the most FIRE-RESISTING and SOUND-PROOF he dormitories of the New England Conservatory of Music, in which prac-y every room is a music-studio.* The various partitions were rated by ssor Norton as shown in the following table:

* The results of these tests, with a description of the partitions, were published in ance Engineering for August, 1902.

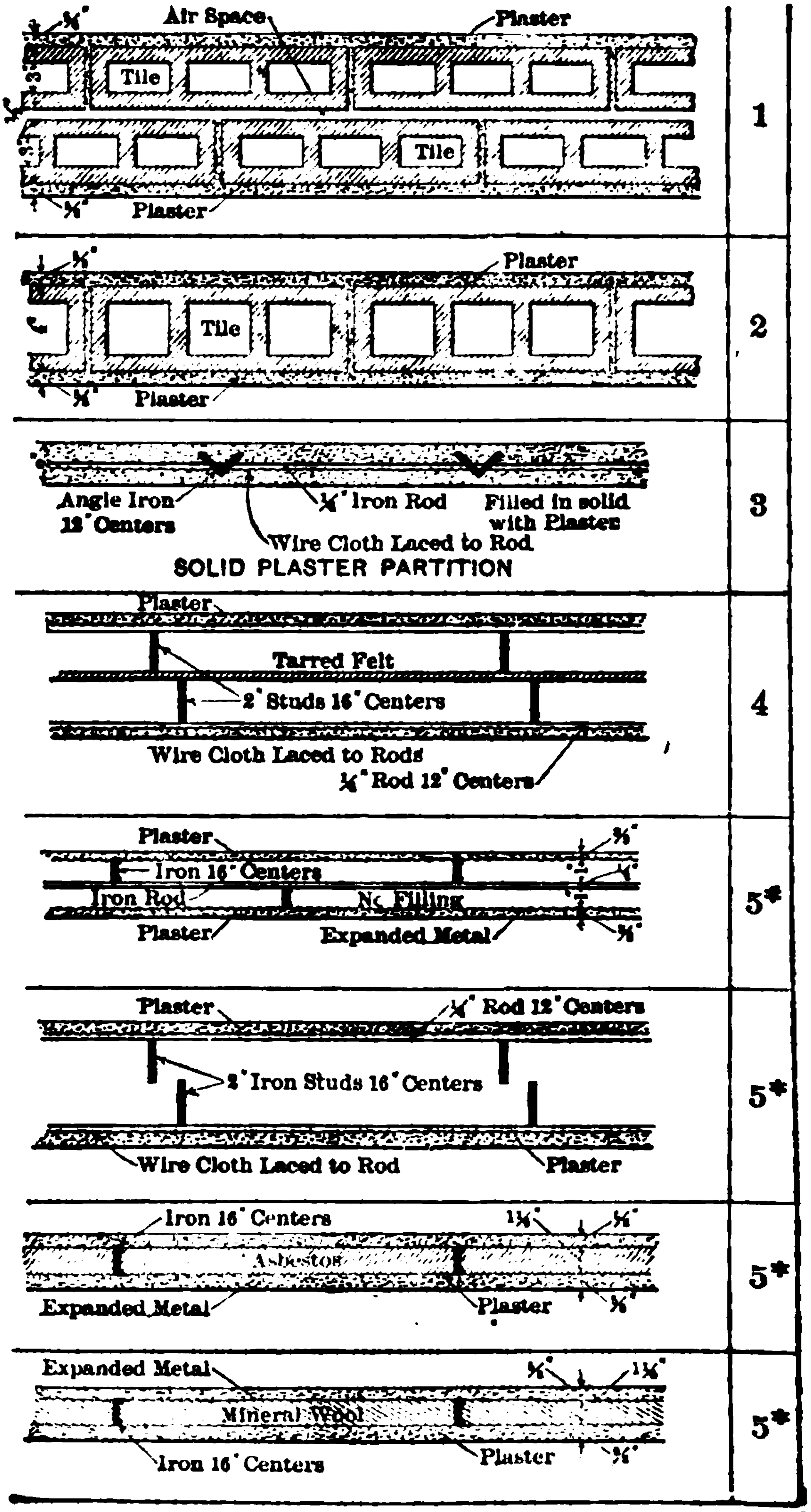


Fig. 76. Relative Deadening Properties of Partitions

Table XVI. Sound-Deadening Properties of Partitions

Room	Side	Scale	Composition
E	Left	100	Cabot's quilt, 3 thick and metal lath
E	Right	95	Cabot's quilt, 2 thick and metal lath
E	Rear	95	Cabot's quilt, 2 thick and metal lath
C	Rear	85	Sackett board, 2 felt on channels
C	Left	85	Sackett board, 2 felt on channels
C	Right	80	Sackett board, 2 felt
D	Rear	75	Metal lath and paper
D	Right	75	Metal lath, paper, and felt
B	Right	60	Two 2-in Keystone blocks with 2-in air-space
A	Rear	50	4-in National terra-cotta blocks
B	Rear	50	3-in Keystone blocks
A	Right	45	3-in National terra-cotta blocks
B	Left	40	2-in Keystone blocks
A	Left	40	2-in National terra-cotta blocks
D	Left	30	2-in metal lath, solid plaster

thing more is to be inferred from the numerical efficiencies, under 'scale
iat the first partition is about three times as good as the last, and that th
cal interval between any two partitions on the list merely indicates th
f the magnitude of the difference between the partitions." Professc
recommended a partition of Sackett Board and plaster with two thick
of Cabot's quilt between the plaster-boards, and this construction wa
l. The studding was put up the same as for the 2-in solid partition, th
cured to each side of the studs, and the plaster-board wired on to th
rough the quilt. This makes as light a partition, also, as it is possible t
ct. The investigations by Professor F. R. Watson of the University c
showed that 2-in solid, metal-lath partitions are more sound-deadenin
n hollow, gypsum-block partitions. Gypsum tile has been found to b
tisfactory than terra-cotta tile of the same thickness.

ng for Outside Walls. The outside walls of fire-proof buildings ar
y finished on the inside by plastering applied directly to the masonry
e walls are of brick, it is often desirable to fur them so that there will b
pace between the plaster and the masonry to prevent the passage c
:. This furring should be either of terra-cotta or metal, and never c
For this purpose furring-bricks may be used. They are made of brick
of the same size as common bricks; but they are hollow. They ar
with the rest of the wall, on the inside face, and bonded into the wa
sual header-courses. Split furring-tiles, also, are often used on the inne
rick walls, as shown in Fig. 77. The tiles are either 1½ or 2 in thic
y 12 in on the face. The face is grooved to give proper bond for th
g. At recesses in the walls partition-blocks are substituted across th
, making a continuous wall-surface. When using furring-tiles, th
ould be careful not to drop mortar into the hollow spaces. When wal
d or lined with tile, solid porous terra-cotta blocks should be built i
nailings are required for bases, picture-moldings, etc. Wire lathin
1 1-in V ribs woven in every 7½ in, makes a good furring for bric
it is easily applied and leaves air-spaces between the wall and plaste
se devices also protect the walls from being warped by heat during
prevent the passage of heat through the walls in summer and winte

Metal Furring. To produce architectural forms in the interior decorati of fire-proof buildings, METAL FURRING and METAL LATH are now almost un versally used. The furring is always of a sham nature, and never employ

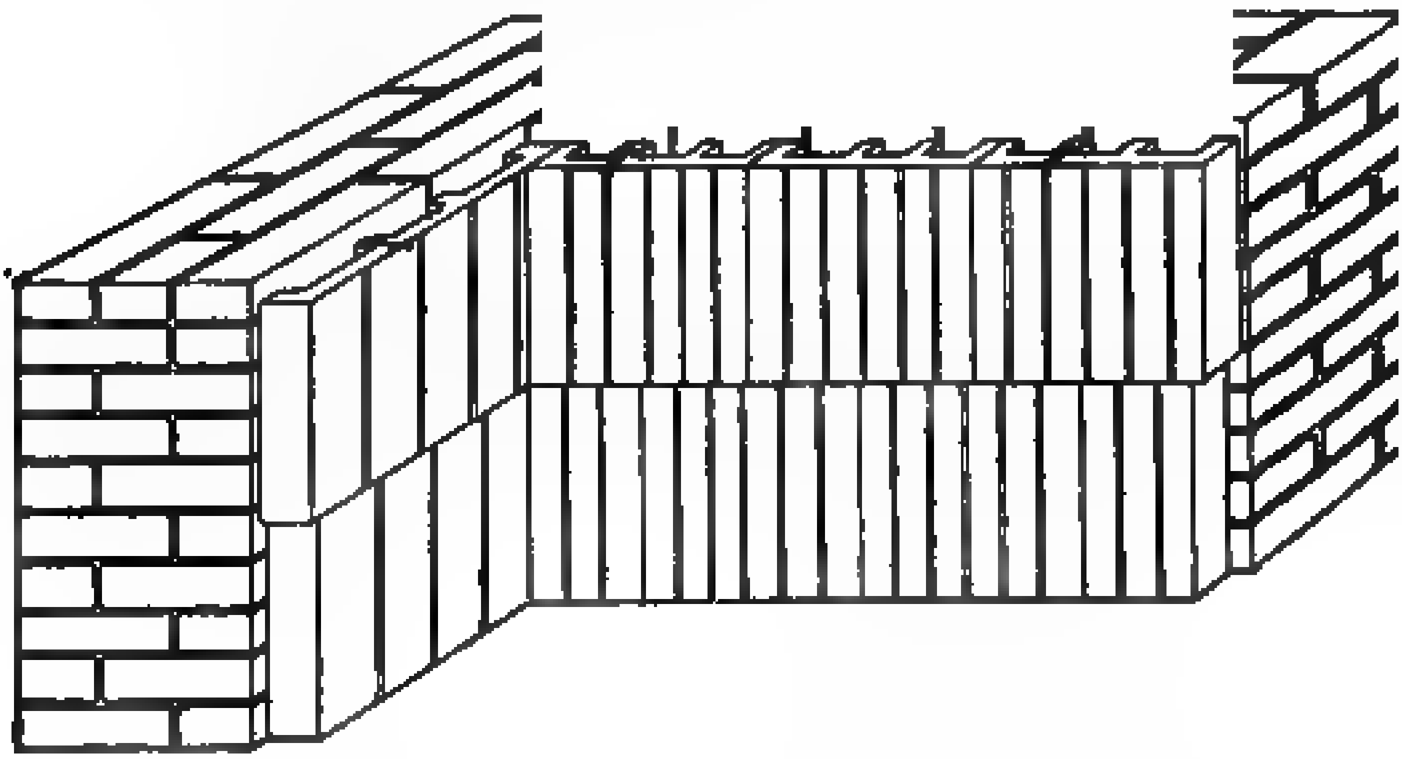


Fig. 77. Hollow-Tile Wall-furring

to carry loads of any magnitude; so that the only requirement is that it sh be incombustible and furnish a satisfactory ground for attaching the metal la For coves, cornices, false beams, etc., the furring-members are made of li bars, angles, tees, or channels, attached to the walls by means of nails, stapl or toggle-bolts, and to the steel beams by means of bolts, hangers, clips, e The furring-pieces are bent or shaped to the approximate outlines of the finish plaster-work, so that when the lathing is applied it will require not more th 1½ or 2 in of plaster to give the desired outline. For plane surfaces, the furri should be brought to within ¾ in of the plaster-line. Deep beams, etc., shou be braced by diagonal rods, to prevent distortion. All structural-steel memb should always be fire-proofed back of the furring. The lathing is secured the furring by means of No. 18 galvanized lacing-wire. The spacing of t furring should be either 12 or 16 in, according to the kind of lath that is to used. When chases in walls are covered over, the covering should be done w metal furring and lath. The casings for vertical pipe-lines, also, should be this construction and the space about the pipes at the floor-level should be fil solidly with fire-proof material, to cut off all connection between stories.

7. Fire-proof Flooring

Fire-proof Flooring. The floor-surfaces of most fire-proof buildings con of hard-wood flooring secured in the usual manner to nailing-strips embedd in the concrete or in the filling above it. It is sometimes advisable to use combustible flooring. The New York City Building Code requires that in buildings over 150 ft in height, the floor-surfaces shall be of stone, ceme tiling, or similar incombustible material, or of wood treated by some proc which renders it fire-proof. For warehouses and factories, floors finished w Portland-cement mortar are about as satisfactory as floors with any other ot floor finish; and cement floors have been much used for the guest-rooms

a. In the latter rooms, the floors are covered with carpets, which are ed to wooden strips embedded in the cement around the borders of the s. This makes a very sanitary floor, and one as easy for the feet as a car-l wooden floor. For public corridors, banks, lobbies, toilet-rooms, etc., the istic, vitreous, ceramic, or marble tilings are generally used. In France and any large quantities of cement tiles are used. Cement tiles have been in-iced into this country, also, but have not yet been able to compete with ncaustic tiles. In most buildings, however, the use of stone, cement, or tile ng is inadvisable. These materials are cold and trying to the feet. As a cement floor-surfaces do not wear well. Asphaltic flooring is sometimes but it is not pleasing in appearance. This material and different floor-ire discussed on pages 1604 to 1609. The characteristics of fire-proofed and its availability for this purpose are considered in the discussion of that ial on page 820.

mposition Flooring. Several attempts have been made to obtain a ig-material which could be spread, without joints, over an entire floor, t the same time be elastic, wear well, withstand water, acids, etc., and not expensive. Various mixtures of magnesite, asbestos, fine sand, sawdust with linseed-oil, and some binder like chloride of magnesium, have been the market under different names, all more or less meeting the require-above stated and being, also, fire-proof. These materials are shipped in rm of a dry powder to the place where they are to be used, and are there with a specially prepared liquid. The resultant is a plastic material is laid upon the surface to be covered in much the same way that ordi-ement or plaster is put on. The materials harden in from 12 to 24 hours lerately dry weather, when the floor is ready for use. When properly e floor presents a smooth, fine-grained, and continuous surface, resem-inoleum. These materials are made in various colors, such as red, white, brown, gray, black, blue, and green, and can be laid on wood, stone, e, asphalt, cement, or metals. Another advantage is that they can be up on the walls so as to form a covered base, without cracks or joints. the manufacturers furnishing such floorings may be mentioned: General lite Company, Long Island City, N. Y.; Marbleoid Company, New York Franklyn R. Muller & Company, Waukeegan, Ill.; and Ronald Taylor ny, New York City.

alt Mastic Flooring. This flooring is in the nature of an ASPHALTIC TE consisting of natural asphalt and a well-graded mineral aggregate of ravel, and crushed stone, ranging in size from that which passes a 200-reen to ⅜ in. The material is sent to the building-site in blocks and is roken up, reheated, and mixed with the coarser aggregate, the softened ing laid down in one or two courses, depending upon the thickness and smoothed down by rubbing with wooden floats. It is laid without tion-joints, the usual thickness being 1½ in, weighing 18 lb per sq ft. shed flooring is tough, ductile, water-proof, resistant to acids, alkali and re-proof, noiseless, and easy on the feet. It is especially suitable for and warehouses. It is manufactured by the H. W. Johns-Manville y, New York

8. Interior Finish and Fittings

or Finish. In buildings in New York City in which the flooring must ombustible material, the interior finish, also, including the doors, door-indow-frames, sashes, bases, and trims, must be made of incombustible

materials. The same materials that are accepted for flooring can be used fo this interior finish also. Several of the largest buildings in New York Cit including the Fuller Building, have all the trim constructed of FIRE-PROO WOOD. In the Hotel Gotham, all the doors and interior finish are made Alignum.

Metal Doors, Sashes, Frames, and Trim.* The effort to make the interi of buildings fire-proof has resulted in METAL-COVERED wood, and in door sashes, frames, trim, and moldings of HOLLOW steel or other metal. Many ver large buildings have in recent years been equipped wholly or in part with the products, and the products themselves have reached a stage of great perfe tion of workmanship and efficiency. Several cities in the United States comp the use of these products for certain parts of buildings which are over a certai height; and it is probably only a question of time when other cities will pa ordinances compelling their use. At the present time cost enters largely int the question of substituting them for wood. Among the first attempts in th United States to fire-proof the interior trim of buildings were those made i New York City, about the year 1880, in the form of metal-covered woodwor by the firm of Campbell & Bantossell of that city. About this time, als there were introduced along with various processes of fire-proofing woodwor FIRE-PROOF PAINTS. Later, FIRE-PROOF WOOD was introduced, that is, wo which has the resin and other inflammable components extracted from it, a the fiber left. In the course of a few years the METAL-COVERED-WOOD indust developed to such a stage that it was possible to trim with its products t interior of a building and keep a good appearance. Notable examples are t Manhattan Life Insurance Company's Building and the Barclay Building, an of more recent date, the Metropolitan Tower,† the Fifth Avenue Office-Buildin the Germania Life Insurance Company's Building, and the Vanderbilt Hot all in New York City; the Hoge Building, Seattle, Wash.; the Hall of Recor Los Angeles, Cal.; the Rockefeller Annex, Cleveland, Ohio, etc. The roug unfinished appearance of the STANDARD TIN-CLAD DOOR set men to seeking product for use in interior finish which would lend itself to more decorati effects. The KALAMEIN IRON and other METAL-COVERED work result In the meantime improvements were constantly being made in HOLL SHEET-METAL doors and trim, and from about the year 1903 HOLLOW STE construction for this work came into use. Owing to its generally super workmanship and to the splendid enamel surfaces which can be given it various baking-processes, this type of interior finish has found favor in the eyes the architects and owners of modern offices, and mercantile and public buildin

Kalamein Iron.‡ KALAMEIN IRON is the trade name given to one of t open-hearth sheet-steel products which is covered with a thin alloy of tin a lead in much the same way that galvanized iron by galvanic immersion coated with zinc. "The name CALAMINE (with Galmei of the Germans) commonly supposed to be a corruption of Cadmia. Agricola says it is fr

* For a brief outline of this subject, illustrated with numerous detail drawings, article on Metal Doors, Sashes, Frames, and Trim, by Professor Thomas Nolan, Kidder's Building Constructon and Superintendence, Part II, Carpenters' Work.

† The Metropolitan Tower has a metal-covered trim which is a special bronze-p construction over a wooden core. This was developed by The John W. Rapp Compa afterwards consolidated with The J. F. Blanchard Company into the United States Me Products Company, New York City.

‡ Among the better known manufacturers of metal-covered work, whose doors inspected and labeled by the Underwriters' Laboratories, Inc., are the United St Metal Products Company, New York City and the Thorp Fireproof Door Compa Minneapolis, Minn.

s, a reed, in allusion to the slender forms (stalactic) common in the formation."* The term KALAMEIN is often used incorrectly, by archid others, for any form of METAL-COVERED woodwork, whether the metal copper, or bronze, to distinguish metal-covered from HOLLOW METAL con-; but the term is obviously misleading and causes much confusion. In instances architects have specified KALAMEIN material expecting BRONZE o be used in the covering, whereas the manufacturer's interpretation of ification was that KALAMEIN IRON was intended.

l-Covered Doors, Frames, and Trim. The cores of METAL-COVERED d frames are built up of oak or white-pine strips dovetailed together se to the grain. In gluing up the strips into stiles and rails the grain strip is reversed in order to resist the tendency of the core to twist. s and rails are mortised, tenoned, and box-wedged, and the cores are with asbestos paper or board and enclosed with sheet metal, either steel ay be painted to match a wooden trim, or electroplated with copper, bronze), or solid sheet copper, brass, or bronze. For doors up to 3 ft idth and 8 ft in height, both sides are often made of continuous sheets which have the panels pressed into them by hydraulic pressure and are eam or joint. The metal sheets of the two sides, in one make of door,† to overlap in a depression on the edges of the door and are secured y screws which pass through both face-sheets. The standard thickness or is 2⅛ in. When these doors are more than 3 ft 4 in in width, each nerally made of two sheets which meet over a middle stile and lock with a flush double-lock joint. This makes a double row of vertical

-Covered Window-Frames and Sashes. Window-frames and sashes, door-frames and doors, are made of METAL-COVERED WOOD. Bronze tal usually recommended and preferred although Kalamein iron may tuted when a much cheaper construction is necessary. This cheaper y be painted and will give fair service but it is not recommended. d iron and copper, also, are used. "Window-frames and sashes of or of sheet-metal over wooden cores are principally used for or skylights where the only danger of fire-contact is through arks. They are NON-COMBUSTIBLE rather than FIRE-RESISTING. s are usually of plate glass, especially if KALAMINE trim is used comply with the law in those cities where non-combustible windows , etc., are required in buildings of a certain class or of a height d limits. Previous mention has been made of their efficiency as ted in the burning of the Kohl building in San Francisco, and their n as a substandard protection, has been pointed out; but for efficient nce, KALAMINE windows, especially, are an unknown quantity, as nce offered by the lighter members, such as sash-rails, is questionable. examples of the work present pleasing workmanship and finish. If osition could be used for the body instead of wood, without producing ction harmful to the metal, a superior type of KALAMINE work would h would be of great value."‡

Metal Finish in General.§ The transition from METAL-COVERED

Dictionary of Mineralogy.
hardson seamless door, made by the Thorp Fireproof Door Company, Minne-
.
vention and Fire Protection, by J. K. Freitag.
the better-known manufacturers of hollow, sheet-metal doors, trim, etc., lstrom Metallic Door Company, Jamestown, N. Y.; the National Metallic

wood to HOLLOW SHEET METAL for doors, sashes, frames, trim, moldings, was naturally and easily made and to-day the latter type of construction, expertly carried out, results in details for interior work which are very eff to resist fire and handsome in appearance. It would be difficult to devise structional details which would be more satisfactory and at the same present greater possibilities in the way of elaborate design and high fi and it is on account of all these advantages that this type of construction is in the interior equipment of many of the best examples of fire-resisting b ings, especially for the doors, frames, sashes, and trim of corridors, hallw stair and elevator-enclosures, and even for entire office-partitions. Becau the non-absorbent character of the baked-enamel finish this material is pa ularly sanitary; and hollow metal doors are more easily cleaned than any ot especially if all moldings are omitted and panels made simply as smooth de sions. The thickness of standard hollow metal doors approved by un writers, varies from 1½ to 2⅛ in.

Hollow Metal Doors. The Dahlstrom patent SHEET-METAL DOOR made from two No. 20-gauge-steel plates, one stile and one panel-face formed from each of the sheets, which are connected by interlocking sean opposite sides of the panels and make practically a double door. In const ing the panels they are first lined with a sheet of asbestos next to the ste each side, and the space between is filled with a layer of hair-felt paper, makes a resilient filling that is a non-conductor of heat. The stiles ar hollow, but strips of cork are laid perpendicularly across the center of ea deaden the metallic ring. The panels are then attached to each other to the door by planting on and welding in place properly formed cross-rai the top and bottom, and wherever else they may be desired; the molding coped over the molded stiles at the sides. The top and bottom edges o door are then reinforced with channels and bars, and the doors made perf straight and rigid. The fire-resistance of this construction is increase letting no rivets or screws pass through from one side of the door to the in the exposed parts. The transmission of heat is thus avoided. Whil door is being put together, provision is made for attaching the hardware. the doors have been put together, they are sent to the finishing depart where the steel is thoroughly cleaned from all rust, grease, or other impu They are then given six or eight coatings of enamel, being baked afte application of each coat in large ovens which are heated to 300° F. the final coat of varnish is put on, they are usually rubbed to an egg gloss-finish, equal in quality to any hardwood-finish, and more durable be baked on. The surfaces can be grained to imitate with wonderful exac any wood, such as quartered oak, mahogany, Circassian walnut, etc. doors are to receive glass panels they are provided with detachable mol to hold the glass in place. Doors of the Dahlstrom, HOLLOW METAL typ installed in the corridors and partitions of the Singer Building and tower† the United States Express Building, New York City; the Bell Telephon change Building, Philadelphia, Pa.; the Seventh Regiment Armory, Ch Ill.; the Pontchartrain Hotel, Detroit, Mich.; the Bank of Commerce Bui St. Louis, Mo.; the First National Bank Building, Denver, Col.; an Royal Insurance Building, San Francisco, Cal. In some of the buildings

Sash Company, Chicago, Ill.; the Solar Metal Products Company, Columbus, Ohio the Central Metallic Door Company, Gary, Ind.

* Made by the Dahlstrom Metallic Door Company, Jamestown, N. Y.

† A severe fire in the twenty-sixth story of this tower was effectually confined room in which it originated by the doors of this type of construction.

n the preceding articles, HOLLOW METAL doors, trim, and moldings are anied by bronze or other METAL-COVERED WOOD window-frames, etc.

w Metal Door-Frames, Trim, and Moldings. After the HOLLOW door reached an advanced stage of construction the manufacturers their attention to the problems involved in making metal frames and s. It was found that moldings made by the ordinary HOT-ROLLED were too rough and heavy and required too much labor to smooth h their surfaces; and that those pressed from light-gauge steel by the methods were not clear-cut and definite in their outlines, and were n length and in variety of shapes. Accordingly, what is known as the AWN METHOD of making frames, trim, and moldings, was developed and l, and moldings made by this process are now used for many kinds of work. The cold metal is drawn through special dies to give it the shape and the bright finish is retained. The corners and angles come p and true and the pieces possess much greater strength and rigidity se HOT-ROLLED and several times thicker. There are dies for over a l shapes. Moldings can now be made in lengths up to 40 or even 50 xtra-freight rates and other drawbacks make it inadvisable to ship it s of over 20 ft. Besides the COLD-ROLLED special high-grade steel, onze, and copper are used in their manufacture. The rolled shapes ngles, channels, and Z bars; moldings for bases, cornices, wire-conduits, bs, sash-bars, panels, and glass; picture-frames, door and window-nd trims of all kinds; wainscoting and chair-rails; and numerous miscel-orts. WROUGHT-IRON welded one-piece door-frames are made for use oof partitions. These frames are constructed scientifically of specially ROUGHT IRON in several different shapes. The mitered corners are gether making the frame one solid piece. They are made for any or type of door or partition, require no bracing, and can be fitted with hinges if required.

Metal Window-Frames and Sashes. (See, also, Sheet-Metal for sting Window-Frames and Sashes, page 902). HOLLOW METAL window-d sashes, as well as those which are made of METAL-COVERED WOOD t iron, wrought iron, drawn bronze, cast bronze, etc., and glazed with , prism glass, electroplated glass, etc., are used in those parts of build-ich the exposure to fire is not great enough to require the use of hinged shutters, or where a more pleasing appearance is demanded than that rom the use of hinged or rolling fire-shutters. Owing to many im-s made in recent years, both in design and details of manufacture, SHEET-METAL window-frames and sashes are now ranked among the of those of moderate cost for general use. The National Fire Pro-sociation, by its recommendations and standardizations, and the labeling systems of the underwriters' laboratories, have been largely al in bringing about these improvements and results. About the vantage connected with the use of SHEET-METAL windows is a relatively rioration when neglected. The materials used for making HOLLOW dow-frames and sashes are galvanized iron or steel; copper; sheet metal, ed; and sheet metal, bronze-plated. The sashes are glazed with LAZE wire-glass where good appearance is an essential requirement, or D OR ROUGH wire-glass where a translucent material only is desired. lear glass, unwired, may be used when additional fire-resistance is not

The National Board of Fire Underwriters fix, within certain limits, constructional details, the maximum permissible sizes of openings for

glass, etc. The principal regulations have been very conveniently cond by Mr. J. K. Freitag.*

Solid Steel Windows. Where large window-surfaces giving maximum are desired, as in factories, the so-called SOLID STEEL WINDOWS are frequ used. They have been given this name because the frames and muntin made of solid, rolled-steel sections, jointed at their junctions or intersectio special methods, in some cases oxy-acetylene welded, so as to make strong stiff frames. The manufacturers generally carry stock sizes varying in ap mate widths from 3 to 6 ft, and approximate lengths from 3 to 9 ft. The panes are about 12 by 18 in. The movable sections, or VENTILATORS, are pi on horizontal axes, though a COUNTERBALANCE TYPE, also, is made for u hospitals and public buildings. Ventilators should not exceed 5 ft in direction, nor more than 18 sq ft in area. Among the principal makers a Detroit Steel Products Company (Fenestra), Detroit, Mich.; David Lu Sons Company, Philadelphia, Pa.; American Steel Window Company, Ch Ill.; and Truscon Steel Company, Youngstown, Ohio.

Electroplated Trim. This product is made by a process which consi electrically depositing a layer of copper on the outer surface of wooden mol or doors. The metallic deposit preserves the markings of the grain of the and makes a very presentable door A good sample of this work has be stalled in the United Engineering Building, New York City, by the New Central Metal Company of the same city. Some very fine work of this ki been done by the Hecla Iron Works of New York City, by electroplating fire-proof material known as Lignolith.

Cement Trim. Keene's cement has been used for many years for ru base-moldings, door and window-trim, etc., and in many European bui practically all of the interior finis this material. Any molding can b in it with good sharp angles, and it i ficiently hard to stand ordinary u Fig. 78 shows a door-opening with of Keene's cement. This detail c further improved by covering the w frame and door with thin metal. metal and cement can be painted sired.

Molded Hollow Tiles for I Finish. These are also being subst for the ordinary wooden finish. Amelia Apartments, erected by Camp at Akron, Ohio, in 1901,† i almost entirely of hollow tile. "The bases, the picture-moldings, an architraves around the doors were made of specially formed tiles, as in Fig 79. These tiles were afterward painted to harmonize with the s of color-decoration. All of the floors throughout the building are covered a cement composition composed of Sandusky cement and ground wood, tro down smooth and level."

FIG. 78. Door-jamb with Cement Trim

Metallic Furniture and Fittings. In offices, banks, libraries, and buildings, the furniture and fixtures are about the only articles on w

* For the principal regulations, conveniently condensed, see Fire Preventio Fire Protection, by J K Freitag.

† Described in the journal, Fireproof, July, 1903.

ted, if the building itself is fire-proof, and if these are made of incom- materials there is no chance for a fire to gain headway or to do much .

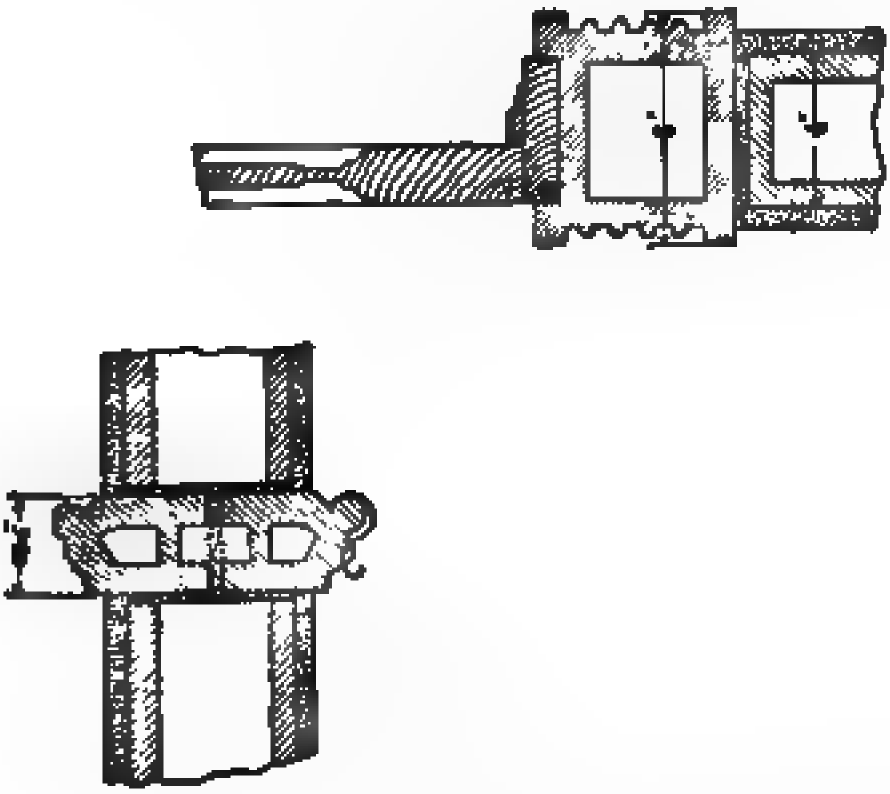


FIG. 79. Hollow-tile Door-trim, Picture-molding and Base

Almost anything in the way of furniture and fittings, including even desks and highly ornamental cabinets, may now be obtained in metal; y libraries, banks, and court-houses have been fitted up and furnished with incombustible cabinet-work. Catalogues can be obtained from ng companies engaged in ufacture of METAL FUR- such for example as the d Construction Company, n, N. Y., the Berger turing Company Canton, e Van Dorn Iron Works, d, Ohio, and the Library New York City and Bos- n.

FIG. 80. Hollow-tile Steps for Staircase

. In a majority of fire- ldings the architects have d themselves with putting BUSTIBLE STAIRS of iron, haps slate or marble treads. As pointed out in the first pages of this unprotected iron cannot be considered fire-proof, but it is difficult to e ironwork of a stairway, as it is usually built, and at the same time an ornamental effect. If exposed metal construction is to be used, cast uch to be preferred to steel, as the cast metal will retain its shape under at far better than thin facings or frameworks of steel. Slate and marble d platforms, unless supported underneath, should never be used in construction. When subjected to heat, marble and slate crack and fall ving the stairs impassable. A fire-department captain in New York his life through the collapse of a marble platform. If these materials used, therefore, there should be a subtread of iron or concrete beneath , really FIRE-PROOF STAIRCASE should be constructed with as little

ironwork as possible, and what ironwork there is, incased in FIRE-RESIS materials. It is possible and practicable to build stairs of clay tiles, brick reinforced concrete, that are absolutely fire-proof. The stairs in the Pe Building at Washington, D. C., are built of brick, with the exception o treads, which are and in many of earlier governm buildings the are of stone. S suitable for s however, are no resistant as cast to heat. Part Building Constru and Superintende contains descrip and illustration brick stairs. The tavino Company built several cases according system of constru using flat clay til bedded in cement, iron-work whate used in this struction; hence eminently fire- Fig. 80 shows tial section of a tile staircase such as was used in the Amelia Apar Building, Akron, Ohio. The blocks were of hard-burned material, and 4 ft long. They were supported upon the partition-walls and were by the mechanics for carrying up material during the erection of the

FIG. 81. Reinforced-concrete Stairs, Government Printing Office, Washington, D. C.

FIG. 82. Ferroinclave Foundation for Stair-treads and Risers

ing. Reinforced concrete, with slate or marble treads, is a good material construction of stairs, and permits of very elaborate and complicated co tion. Fig. 81† shows the construction of the stairs in the Government P

* By Frank E. Kidder, rewritten by Thomas Nolan.
† From the Engineering Record, of Dec. 6, 1902.

at Washington, D. C. These stairs have steel girders and strings enclosed solid concrete, which is molded to form the steps and risers, as shown in the The steel strings, however, are hardly necessary, as the reinforcing-bars ifficient strength. Some excellent details for ornamental iron stairs were hed in Fireproof, March, 1903, in an article by J. K. Freitag. The cor-d sheet metal, known as Ferroinclave (page 851), offers a very conven-oundation for cement stairs. When built between walls or partitions h an open string, Fig. 82 shows one way in which the material has been the stairs being finished with about 2 in of cement over the metal and ed underneath. The Ferroinclave is bolted to lugs or brackets screwed ast on the strings. Slate or marble treads and risers may be embedded mortar if desired. (See, also, pages 947 and 983.)

9. Protection from Outside Hazard

ndow-Protection. To be thoroughly protected against the outside , buildings must have the openings in the outside walls provided with some of effectively closing those openings against flame. The same provision be made for openings in the partition-walls of large buildings. Four L TYPES of devices are in use for this purpose: (1) tin-covered wooden s; (2) steel shutters or doors; (3) metal frames and sash, glazed with ass; and (4) water-curtains.

s of Window-Protection Compared. When properly constructed, -COVERED WOODEN SHUTTER is still the most effective window-protection. ery severe fire in Lynn, Mass., in which the heat was intense enough to ost of the tin from the outside of the tinned plates covering the shutters, found afterward that the wood was charred to a depth of only about The shutters were warped slightly, but afforded sufficient protection the heat to allow men to remain behind them to put out such fire as nally crept through. This would not have been possible behind iron s under similar conditions."* STEEL SHUTTERS, under the action of arp very readily and transmit considerable heat. They belong to the t type of window-protection. "There is one objection to the use of s on window-openings, and that is that they depend on fallible human to be effective. They must necessarily be open while the building is When the need for them comes they are apt to be overlooked and are not Certain it is that on many buildings they are not closed at night."* TAL-FRAME-AND-WIRE-GLASS WINDOWS are not as unsightly as shutters st any kind are apt to be. They are more likely to be closed at night re readily closed when necessary. They do not hide a fire and are sily opened when it is necessary to reach a fire. The one serious objec-them is the intense radiation of heat from the wire-glass.†

overed Wooden Fire-Shutters and Doors. The effectiveness of vice depends on its construction. "Only well-seasoned non-resinous dressed, tongued and grooved in narrow boards, should be used. Wood ing moisture or resin may generate, under heat, sufficient steam or gas off the tin covering and expose the wood to the flame. The body of r should consist of two or three layers of such boards laid at right-angles ch other and fastened together by clinch-nails. The best grade of tin be used. No solder must be used, and the tin plates should be lock-

* Insurance Engineering, Dec., 1902.

† For a consideration of water-curtains, see page 903.

jointed, with the nails in the seams. The nails must be long enough, at l
1⅛ in, to secure a good hold beyond the depth to which the wood is likely
char, which is about ¾ in. Under intense heat the wood is certain to char,
if the nails are long enough to hold the tin up against the wood, and the
is properly put on so as to keep the air out to prevent burning, the shu
will stand under severe strains."* The hinges, fastenings, or hangers mus
bolted to the door, not nailed or screwed, as nails or screws would pull
during a fire. If hung on hinges, the hinge-hook should be built into the w
This door was designed for use in mills, but it has worked so satisfacto
that it is generally adopted wherever a fire-proof door is wanted and its
pearance is not objectionable. Fire-proof shutters, also, are made in this w
The National Board of Fire Underwriters issues complete specifications†
this type of door and shutter, and these specifications should be closely
lowed for satisfactory results. Doors of this type, provided for the openi
in interior partition-walls, are often, and wherever possible should be, hung
inclined tracks so that they will close automatically. Where it is desirabl
keep them open most of the time, an automatic release operated by a fus
link is provided. (See, also, page 778.)

Metal-covered Wooden Doors as Fire-Doors. Wooden doors cove
by the KALAMEIN or other process (page 894) are sometimes used as fire-d
where appearance is a consideration. They are not considered equal, howe
to the STANDARD TIN-COVERED WOODEN DOORS.

Steel Fire-Doors and Shutters. For a satisfactory STEEL FIRE-D
a ⅛-in sheet of steel should be used, and it should be reinforced on the b
with a frame of angle-irons, not less than 1½ by 1½ by ¼ in, and increasin
size with the door or shutter. These doors or shutters may operate in on
three ways: (1) swing on hinges, (2) slide on tracks, or (3) roll vertica
The SWINGING DOORS or shutters are the most reliable as there are no c
plicated parts to get out of order. They should be hung on eyes built into
masonry walls. SLIDING DOORS OR SHUTTERS must have the rails on which t
operate protected by metal shields to prevent obstruction. For larger openi
the ROLLING SHUTTERS are generally preferred. They are made in horizo
jointed sectional strips, which wind up on a roller placed in a pocket above
opening, the ends moving in metal grooves to hold them in place. They g
erally operate VERTICALLY, although some are made to operate HORIZONTAL
the rollers being set vertically in pockets at the sides of the openings. Th
latter are more apt to get out of order. The VERTICALLY operated door
shutters are balanced by springs or weights to make them move easily up
down. Where they are intended to be closed in case of necessity only, t
are slightly weighed and held open by means of fusible links, so that in cas
fire they will close automatically.

Sheet-Metal for Fire-resisting Window-Frames and Sashes.‡ Th
are now made weather-tight and perfectly practicable in all respects, and sh
be used wherever fire-resisting windows are desired. The sashes are m
especially for holding wire-glass. These SHEET-METAL WINDOWS are mad
a great variety of forms to meet all purposes and the sashes may be station
pivoted either horizontally or vertically, hinged, or double-hung with weig
like ordinary windows. For factories, warehouses, stairways, and elev
shafts, a stationary lower and a pivoted upper sash are commonly used, as t
is the cheapest type of window. The double-hung windows are now m

* Insurance Engineering, Dec., 1902.

† To be had for the asking.

‡ See, also, Hollow Metal Window-Frames and Sashes, page 897.

ork as smoothly as wooden sashes in ordinary box frames. For offices, ls, etc., a window having two sashes, glazed with wire-glass, and closing locking automatically in case of fire, and a third inner sash glazed with glass, has all of the advantages of an ordinary window with the additional ntages of fire-protection and better diffusion of light. Metal fly-screens can be used with these windows. All movable sashes, glazed with wire-, should be provided with a device by which the sashes will close and lock matically in case of fire. Two thicknesses of wire-glass are sometimes used a ventilated air-space of at least 1 in between the lights.

10. Extinguishing Devices and Precautionary Measures

ater-Curtains. "The vulnerable portion of buildings generally is the , where great window-openings are desired for purposes of light, and where considered objectionable on account of appearance to have shutters or wire-glass windows. These large window-openings afford great oppories for the spread of fire across streets. The danger of damage is much ased where the fronts, as is very common, are made of unprotected metal. A notable example, illustrating such danger, was the building of the hattan Savings Institution, New York City, which was severely damaged almost destroyed by a fire in a six-story non-fire-proof building across the t. Such conditions might be overcome to some extent perhaps, by the duction of some system such as the WATER-CURTAINS that were placed on hicago Public Library. This is practically a SPRINKLER SYSTEM set along dge of the cornice of the building, and so arranged as to furnish a thin of water in front of the building. Such a sheet will, however, not extend fore it is turned into spray and thus becomes practically useless. A similar gement placed at each window-opening might be more useful, though it btful whether it would be of much value in any severe conflagration."* rules of the National Board of Fire Underwriters for OPEN SPRINKLERS ter-curtains determine the sizes of piping and feed-mains, and the general gement of the system.

ecautionary Measures in General.† No matter how thoroughly a build- fireproofed, if it is filled with combustible goods, as a warehouse, store, or y, there is always the possibility of a fire, which, if unchecked when first d, must necessarily entail a great loss and more or less damage to the ng. If a fire is discovered and checked in its incipient stage this loss is ed. There are now many valuable devices for DETECTING and CHECKING which should be installed in every warehouse, and which often may be l with advantage in buildings used for other purposes. The more important se are automatic alarms, automatic sprinklers, and standpipes.

tomatic Alarms. The prompt discovery of fire generally brings about ot extinguishment, but as it is not practicable to have someone on duty in rts of a property at all times, fires may gain serious headway before being ered, unless some system of automatic notification is used. Next to auto- sprinklers, approved automatic fire-alarm systems, THERMOSTATS, are per- the most important of the fire-protection devices. There are two general s of thermostats: one which operates at a FIXED OR PREDETERMINED RATURE, and the COMPENSATING-TYPE. The latter requires a certain rise perature within a given time. This latter type is common in Europe, while s country the fixed-temperature-type has been preferred. The compen-

* Insurance Engineering, Dec., 1902.

† See, also, Chapter XXII, page 768.

sating-types seem to have been used with some success in certain sections, have not proved altogether satisfactory. For general use it would appear the SOLDER-TYPE OF THERMOSTAT has many advantages when used in connect with a simple CLOSED-CIRCUIT SYSTEM. The most common type of thermos system is the ELECTRIC SYSTEM, in which the thermostats are designed to o or close the electric circuit and cause bells to be rung at designated points. thermostats, or circuit-closers, may be of the fixed-temperature type or adjus to operate at any desired temperature. The former are chiefly of the SOLD TYPE, while the most common variety of the latter type consists of a SPRING TWO DISSIMILAR METALS, which expand unequally.

The Aero System. The best-known system of the compensating-type is AERO SYSTEM installed by the Aero Fire Alarm Company, New York Cit This consists of a small copper tube attached to the ceiling. A quick rise temperature, as in the case of fire, expands the air in the tube and acts on a sen tive diaphragm, which latter makes an electrical connection, causing a tra mitter to operate and send in an alarm.

The Reichel System is installed by the Pacific Fire Extinguisher Compa San Francisco, Cal. This system is of the COMPENSATING-TYPE, the REICH THERMOSTATS* consisting of a THERMOPYLE of special design which is connect in series with an electric circuit. Any rapid increase in heat generates suffici current to actuate a transmitter. Slow changes in temperature do not oper the system.

The Derby Automatic Fire-Alarm System is installed by the Americ Fire Prevention Bureau, New York City. This system consists of a TWO-WI CLOSED CIRCUIT, and uses the DERBY FIRE-SENTINELS,* in multiple, across line. Any derangement of the circuit gives a local or central station troubl alarm. Upon the operation of a SENTINEL THERMOSTAT, resistance is au matically cut out of the circuit, thereby causing the operation of fire-gongs a transmitters. The Derby Fire Sentinel can be used on wiring systems utiliz primary, storage, or public-service energy up to 110 volts. The Sentinels a made for attaching to open wiring and also for use in connection with conceal work.

The Watkins Thermostat is installed by the Automatic Fire Alarm Co pany, New York City. It consists of a perforated metal case, enclosing a FL SPRING OF TWO DISSIMILAR METALS. The spring is fastened at one end, and t heat causes a movement, due to the UNEQUAL EXPANSION OF THE TWO METAL WATKINS THERMOSTATS are wired in multiple, the wiring system being part op and part closed. The thermostats are adjusted by hand. They are likely to affected by corrosive influences, moisture, and rough handling. This syste however, has been more largely installed than any other, being the princip type of thermostat used in Boston, New York, and Philadelphia, where, wi good supervision, its record has been satisfactory.

Automatic Sprinklers. "An AUTOMATIC SPRINKLER is a device for distribu ing water by means of a valve which is arranged to open under the action heat, as from a fire which it is intended to extinguish. The distribution water which results from properly located sprinklers occurs in the form of rain of jets or drops, and is sufficient to drench almost any inflammable sto beyond the point of ignition. The distribution is also economical, as the wat is more evenly applied than from a nozzle attached to a fire-hose, and the sour is directly above the fire. Whenever combustible merchandise constitut the contents of a building, AUTOMATIC SPRINKLERS are of great value, and i

* Approved by the Underwriters' Laboratories.

ings of a height so great as to make the upper stories difficult of access, ially if containing large areas and very combustible contents, sprinklers itute the best protection obtainable."* SPRINKLER-SYSTEMS may be divided wo general types: (1) the WET-PIPE SYSTEM, or automatic sprinklers, just ibed; (2) the DRY-PIPE SYSTEM. Where the water cannot be kept from ng in the ordinary wet-pipe system, recourse is had to the dry-pipe system. sprinkler-pipes are filled with air under pressure, which is automatically ed by the opening of a HEAD-VALVE under heat. This release of pressure s the DRY VALVE in the main supply-pipe, allowing water to flow through prinkler-pipes and the open heads. Automatic sprinkler-heads are made pen at various temperatures: ordinary, 155° to 165°; intermediate, ; hard, 286°; and extra hard, 360° F. The higher-temperature sprinklers ut in locations where the heat is above normal, such as boiler-rooms and ooms. Various types, made by the following manufacturers, have been oved by the National Board of Fire Underwriters:† International ikler Company, New York City; General Fire Extinguisher Company, idence, R. I.; Automatic Sprinkler Company of America, New York City; der Brothers, St. Louis, Mo.; Esty Sprinkler Company (H. G. Vogel pany, New York, sole agents); Globe Automatic Sprinkler Company, delphia, Pa.; Independent Aetna Sprinkler Co., Philadelphia, Pa.; Ohio matic Sprinkler Company, Youngstown, Ohio; and Rockwood Sprinkler pany, Worcester, Mass.

rinkler Supervisory Devices. These devices consist of apparatus for NSMITTING SIGNALS when gate-valves are closed or open; when water in s falls below or is restored to a predetermined level; when pressure in nks falls below or is restored to a predetermined amount; when water ıks falls below or rises above predetermined temperatures; also to transmit r-flow signals and to withold signals from water-surges or variable pres-" They are used in connection with CENTRAL-STATION SIGNALLING ms for supervising the operation and maintenance of sprinkler-equip-s. The devices of the American District Telegraph Company of New City, are approved by the National Board of Fire Underwriters.†

nd-Pipes and Hose-Reels. In office-buildings, hotels, and apartment-s, where sprinkler-systems are hardly suitable, STAND-PIPES with HOSE- in each story and on the roof, ready for instant use, constitute the best s of quickly controlling a fire. All buildings over certain heights should equipped, the height being fixed by the ability of the local fire department ch effectively the upper parts of the building with its hose-streams. The pipe should be from 2½ to 6 in in diameter, according to the size and of the building, and should be connected with the water-supply of the ng and provided with Siamese connections at the street-level for the fire ment. Check-valves should be provided, so that when the fire-depart-ngines are attached, their force will be added to the force due to the head er from the fire-tanks, or to the fire-pumps, or to the force of the city system. Stand-pipes should be placed within the stair-enclosures. In ities the practise is to attach them to the outside fire-escapes of the g. The number and location of stand-pipes should be such that all parts building can be reached by at least one stream supplied by hose not ng 100 ft in length.

* J. K. Freitag.
† List of Fire Appliances, National Board of Fire Underwriters.

CHAPTER XXIV

REINFORCED-CONCRETE CONSTRUCTION *

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1. Introductory Notes

Definition. The term REINFORCED CONCRETE is defined in the propose standard regulations of the American Concrete Institute as "an approved mi ture of Portland cement, with water and aggregates in which metal (generall steel) has been embedded in proportionately small sections, in such a mann that the metal and the concrete assist each other in taking stress."†

Historical Notes. The great value of concrete as a structural material whe subjected to compression only has been recognized for centuries. The use reinforced concrete, however, as a practicable and commercial form of constru tion is comparatively recent. It is true that as far back as 1869, Franço Coignet of Paris took out letters patent on a combination of iron and concret and that even before this, in 1867, the principle of reinforcing concrete with iro had been applied by P. A. J. Monier, a gardener of Paris, to the making of larg flower-pots; still, the general application to building-construction did not occu till about the middle of the last decade of the nineteenth century. In its develop ment it was first applied to bridge-construction. The discussion of the subjec in this chapter is confined to its use in the construction of buildings. Th earliest example of a building of reinforced concrete in this country, and probabl in the world, is that erected in 1875 by W. E. Ward, near Port Chester, N. Y. in which "not only all the external and internal walls, cornices, and towers we constructed of concrete, but all of the beams and roofs were exclusively made o concrete reinforced by light iron beams and rods." ‡

The Erection of Reinforced-Concrete Work. In general outline, a build ing operation in reinforced concrete consists in the usual preparations of the sit by excavation or otherwise, the provision of suitable foundations for walls columns, or other supports, the erection of a series of wooden molds or forms, th placing of the necessary steel reinforcement, the pouring of the concrete, an the removal of the forms after the concrete has set sufficiently to sustain itsel and the load that may come on it during construction. From the beginnin of the erection of the forms the successive steps are progressive, that is, th placing of the steel and pouring of the concrete are going on in the lower sec tions or stories while the forms are being erected for the upper sections or storie So that in a large operation the carpenters, the steel-setters, and the concreter may all be working at the same time, one set slightly in advance of the others without interference one with the others. These several steps in the operatio

* For Concrete in general and Mass-Concrete, see Chapter III, pages 240 to 251; fo Strength of Concrete without Reinforcement, Chapter V, pages 283 to 287; and fo Reinforced-Concrete Factory-Construction, Chapter XXV. See, also, Chapter XXII pages 817 and 842.

† Proc. Am. Concrete Inst., Vol. XV, 1919.

‡ For a further and more extended history the reader is referred to the larger treatise on this subject and to Edwin Thacher's article in Engineering News, March 26, 1903.

nsidered in greater detail in Chapter-Subdivision 7, page 962, Erection of rced-Concrete Construction.

2. Materials Used in Reinforced-Concrete Construction

Materials used in reinforced concrete are CONCRETE and STEEL. The te forms the mass of the construction. Its proper use is to resist com-n. While it has some tensile strength the amount is so small and so e that it should always be neglected. STEEL is used for the reinforcing l as it furnishes the greatest amount of strength at the least expense. HT IRON could be used, but it is practically unobtainable under present ons, and, as already intimated, its use is not economical.

rete. The CONCRETE consists of a mixture of cement and some aggre- definite proportions, with the necessary water to cause the setting of the .

ent. PORTLAND CEMENT should always be used in reinforced concrete, hould always be tested before being used. Even in small jobs it is im- to know that the cement is strong and sound. In purchasing the cement, tificate of some reliable testing-laboratory should be made one of the ns of acceptance. Under all circumstances, it is always best to have ing done at some well-established and properly equipped cement-testing ry. The results of tests in temporary laboratories are often abnormal y lead to unnecessary controversies with the manufacturers. To be ole, a cement should meet the following requirements as called for in l specifications.*

FIC GRAVITY. The specific gravity of the cement should be not less o.

ESS. It should leave by weight a residue of not more than 22% on a sieve.

OF SETTING. It should develop initial set in not less than 45 or 60 min- ording as the Vicat or Gillmore needle is used, but must develop final in 10 hours.

LE STRENGTH. The minimum requirements for tensile strength for s of 1-in-square section should be as follows, and should show no retro- in strength within the periods specified:

NEAT CEMENT

's (1 day in moist air, 27 days in water)........... 600 lb per sq in

ONE PART CEMENT, THREE PARTS STANDARD OTTAWA SAND

's (1 day in moist air, 6 days in water)............ 200 lb per sq in
s (1 day in moist air, 27 days in water)........... 300 lb per sq in

ANCY OF VOLUME. Pats of neat cement about 3 in in diameter, ½ in the center, and tapering to a thin edge, should be kept in moist air for a 24 hours. The pat is then exposed in an atmosphere of steam, 1 in above ater, in a loosely closed vessel, for 5 hours.

pats, to satisfactorily pass the requirements, should remain firm and show no signs of distortion, checking, cracking, or disintegration.

e complete standard specifications see the latest Year Book of the Am. Soc. Mats. See, also, Chapter III, page 237, for the principal clauses of the last Specifications for Portland Cement, adopted in 1916 and effective January y the Am. Soc. for Test. Mats. The tensile strengths for neat cement are ed.

SULPHURIC ACID AND MAGNESIA. The cement should not contain more t 2% of anhydrous sulphuric acid (SO_3) nor more than 5% of magnesia (Mg The test for CONSTANCY OF VOLUME OR SOUNDNESS is of particular importa for reinforced-concrete work. When used in large masses an occasional b of concrete made with unsound cement may not seriously affect the final re but in reinforced-concrete building operations, where the different member the structures are comparatively small, the safety of the entire building be jeopardized by the use of a small amount of unsound cement in some im tant part, such as a column.

Aggregate.* By the term AGGREGATE is understood the materials, inclu the sand, mixed with the cement to make the concrete. In practically all c the sand is a necessary element.

Sand. "The SAND should be clean. One may obtain some idea of its cle ness by placing it in the palm of one hand and rubbing it with the fingers of other. If the sand is dirty, it will discolor the palm. Unless from a ban known quality, a sand should be tested for tensile strength of mortar, be using. Preference should be given to sand containing a mixture of coarse fine grains. Extremely fine sand even if clean makes a weak mortar and sh never be used unless with a large excess of cement."† Mortars composed of part Portland cement and three parts fine aggregate or sand, by weight, sh show a tensile strength of at least 70% of the strength of 1 : 3 mortar of the s consistency and of the same cement mixed with standard Ottawa sand. New York Regulations specify that fine aggregate shall consist of sand, cru stone, or gravel screenings, passing when dry, a screen having ¼-in-diam holes, and passing not more than 6% through a sieve having 100 meshes linear inch. The Chicago Regulations specify that not less than 45% shal retained on a screen of 400 meshes to the square inch. (See, also, page 241.)

Coarse Aggregate. For the COARSER MATERIAL OF THE AGGREGATE m materials are used and many others have been suggested. Its selection is erally dependent upon local conditions. If possible, gravel or crushed s should be used. Whatever is used should be a clean, hard substance that secure to the concrete the necessary strength; that is, the crushing stre of this material should be equal to or greater than that of the mortar used least at the age of 28 days. In any case, where no reliable information is t had on the strength of a concrete made from a given aggregate, careful inv gation should be made before such material is used. (See, also, page 241.)

Gravel. GRAVEL, like sand, should be clean. If dirty it should be wa before being used. To get the most satisfactory or uniform results, gr should be screened and graded and then mixed in definite proportions, as RUN OF THE BANK will generally not give uniform results. (See, also, page

Stone. The most satisfactory stone that can be used is TRAP-ROCK (u which term are included most of the rocks of igneous origin), because o toughness and great compressive strength. The GRANITES, as they are c mercially known, are considered by some equal in quality to trap-rock fo making of concrete. The presence of mica in considerable proportion in s of the so-called granites would seem to make them unsuitable. LIMEST

* See, also, Chapter III, pages 240 to 251. The data there on Aggregates, Pr tioning Materials, etc., relate more particularly to mass-concrete, while the da Chapter XXIV is intended to cover, more in detail, reinforced concrete.

† Treatise on Concrete, Plain and Reinforced, Taylor and Thompson, third ed 1916, page 12.

soft varieties are excepted, make excellent concrete as far as strength is rned. They would, however, seem to affect the fire-proof character of oncrete. (See Tables on page 956.) The harder and more compact TONES, also, may be used successfully, but great care must be exercised ir selection. CONGLOMERATE, which is in reality a hard, coarse sandstone, l give very satisfactory results. On account of their low crushing strength, or SHALE should not be used in concrete. Besides the stones thus far ned, broken BRICK, TERRA-COTTA, FURNACE-CLINKER and FURNACE-SLAG een suggested. In the selection of broken brick or terra-cotta, care must en to get hard-burned material. The crushing strength of such material well selected, is a little more than that of acceptable concrete, 28 days But ordinarily, commercial brick or terra-cotta will not meet the require- for a good aggregate, and these materials should be used only as a last and then only after careful investigation. (See, also, page 241.)

ders. FURNACE-CLINKERS should be clean and entirely free from com- le matter. CINDERS are often used where fireproofing is the primary ration, and no doubt good constructions may be obtained, with extreme y the use of clinker or cinder concrete, especially if the material is ground, d and graded as suggested for gravel. But in general practice the con- ; not uniform in quality and is unreliable in strength. It is therefore not red in this chapter. In Chapter XXIII, Fireproofing of Buildings, its liscussed on pages 817 and 818. (See, also, page 242.)

of Aggregate. The SIZE OF THE AGGREGATE may vary from ¼ to 2½ argest diametrical dimension, depending on the particular purpose for t is to be used. Where the mass of concrete is comparatively large the te may run as high as 3 in in size. This may sometimes be the case dations and in large piers and thick walls. In columns, girders, beams bs, very unsatisfactory results would be obtained if so large a stone were For such work no stone or other aggregate should be used larger than pass a 1-in screen. In important girders and columns, especially when forcing-bars are closely spaced, the size should be made even smaller so oncrete of viscous consistency is produced "which will pass readily be- nd easily surround the reinforcement and fill all parts of the forms." * AXIMUM SIZES allowed for the aggregate in reinforced concrete in the dif- ities are as follows: St. Louis and Buffalo, stone that will pass a ¾-in ring, "three-quarter-inch stone"; New York, Cleveland and Philadelphia, at will pass a 1-in ring; Chicago, stone passing 1-in-square mesh; San o, for floors and fireproofing, 1-in stone, for foundations, 2-in stone. so, page 241.)

r. "The WATER used in mixing concrete should be free from oil, acid, or organic matter." *

rtions of the Materials. The proper PROPORTION OF THE MATERIALS into the concrete is dependent upon the size and character of the mate- 1 cities in which there are regulations governing reinforced-concrete con- , the mixture to be used is generally specified. In the absence of other tions the most satisfactory and reliable mixture is, one part of Portland two parts of sand and four parts of stone or gravel. It is the mixture been used in most of the experimental work on reinforced concrete, e is therefore much trustworthy information to be had concerning it. se of large or important operations, however, great economy can often

* Trans. Am. Soc. C. E., 1917, Vol. 81, page 1115.

be effected by a preliminary study of the materials to be used and of th proper proportions. In general, for given materials, the most economical n ture is also the strongest. The old method of determining the proportion concrete by measuring the voids in the coarser particles by means of w poured into a box containing 1 cu ft of the material and then providing t quantity of finer material, assuming the cement the same as sand, is not t recommended. It does not give accurate or satisfactory results. A be method is to take the materials to be used and make trial-mixtures by var the proportions, always using, however, the same amount of cement and w These trial-mixtures are placed successively in a measuring vessel of fixed and tamped, and the height to which the vessel is filled for each mixtu noted. The proportions that give the lowest height, or result in the sma volume, will give the most satisfactory concrete. (See, also, page 242 and lowing pages.)

The best and most scientific method, however, is that known as the MECH ICAL ANALYSIS, devised by W. B. Fuller. In this method the available mater including the cement, are separated into the various sizes by means of a s of sieves; curves are plotted which indicate the percentages of the whole n which pass the several sieves; and from a study of these curves the proport of the different aggregates are determined. For a detailed description of method the reader is referred to the chapter on Proportioning Concrete in 1912 edition of the Treatise on Concrete, Plain and Reinforced, by Taylor Thompson. As an example of the saving possible, the following case, give the work just referred to, will be of interest.

"The ordinary mixture for water-tight concrete is about 1 : 2 : 4, whic quires 1.57 barrels of cement per cubic yard of concrete. By carefully gra the materials by methods of mechanical analysis the writer has obtained w tight work with a mixture of about 1 : 3 : 7, thus using only 1.01 barre cement per cubic yard of concrete. This saving of 0.56 barrel is equiva with Portland cement at $1.60 per barrel, to $0.89 per cu yd of concrete. added cost of labor for proportioning and mixing the concrete, because o use of five grades of aggregate instead of two, was about $0.15 per cu thus effecting a net saving $0.74 per cu yd. On a piece of work involv say, 20 000 cu yd of concrete, such a saving would amount to $14 800 amount well worth considerable study and effort on the part of those in res sible charge."

In the ordinances or regulations governing reinforced concrete of va cities the proportions to be used are generally prescribed. In New York, ' concrete for reinforced-concrete structures shall consist of a wet mixture o part of Portland cement to not more than six parts of aggregate, fine and co either in the proportion of one part of cement, two parts of fine aggregate four parts of coarse aggregate, or in such proportion that the resistance o concrete to crushing shall not be less than 2 000 lb per sq in after hardenin 28 days." In Chicago, various grades of concrete are specified with the mate compressive resistance, to be developed, from a mixture of 1 : 1 : 2 an ultimate strength of 2 900 lb per sq in, to a 1 : 3 : 7 mixture with a stre of 1 500 lb per sq in. In Buffalo and San Francisco the proportion is g as one of cement to six of aggregate; in Boston it is one of cement to fi aggregate.

Compressive Strength of Reinforced Concrete. For reinforced-con work no mixture should be used that does not develop a COMPRESS STRENGTH of at least 2 000 lb per sq in at the age of 28 days. The crus strength of various concretes is shown in the following table:

Table I. Compressive Strength of Portland-Cement Concrete of Different Proportions

Proportions			Age, months	Compressive strength per sq in	Authority
nt	Sand	Stone			
	1	0	4	4 370	James E. Howard, Tests, Watertown Arsenal
	2	0	4	2 506	
	3	0	4	1 812	
	4	0	4	830	
	5	0	4	532	
	6	0	4	169	
	7	0	4	118	
	2	4	4	2 178	
	3	6	4	1 815	
	4	8	4	1 135	
	5	10	4	707	
	6	12	4	738	
	2	2	4	1 768	
	2	3	4	1 911	
	2	4	4	2 147	
	2	5	4	2 452	
	2	6	4	2 124	
	2	7	4	1 650	
	2	8	4	1 295	
	2	4	1	2 399	G. A. Kimball, Tests of Metals, U. S. A.
	2½	5	1	3 255	Taylor and Thompson, Tests, Watertown Arsenal
	3	5	1	2 042	Watertown Arsenal, Tests of Metals, U. S. A.

ing Stresses for Reinforced Concrete. Some formulas for the
of reinforced-concrete construction provide for the use of the ULTIMATE
H of the concrete and the application of a FACTOR OF SAFETY. This
is not to be recommended as it necessitates either the test of the con-
the assumption of an ultimate strength. While it is undoubtedly de-
hat the concrete should be tested, this is generally impracticable when
ling is being designed. It should be done during construction and is
the best work, to make sure that the concrete is up to the require-
Various factors of safety from two and one half to ten have been pro-
Different factors of safety are used for different members of a structure
fferent conditions. This is another reason why it would be better to use
STRESSES than ULTIMATE STRESSES. The following WORKING STRESSES
mended for reinforced concrete that will develop a CRUSHING STRENGTH
b per sq in in 28 days:

e fiber-stress in compression.................... 650 lb per sq in
g-stress.. 40 lb per sq in
shearing-stress when all diagonal tension is re-
by the steel, and the steel-resistance to both
ive and positive moments is fully developed.... 150 lb per sq in
ompression....................................... 500 lb per sq in
ress between concrete and plain reinforcing-bars.. 80 lb per sq in
ress between concrete and suitable deformed bars. 100 lb per sq in

ives the stresses allowed by various building ordinances.

Table II. Working-Stresses for Reinforced-Concrete Construction

Authority	Extreme fiber-stress, concrete in compression, lb per sq in	Direct compression in concrete, lb per sq in	Shearing-stress in concrete, lb per sq in	Shearing-stress in concrete when all diagonal tension is resisted by steel, lb per sq in	Adhesion of steel to concrete, lb per sq in	Tensile stress in steel, lb per sq in	Tensile stress in steel to resist diagonal tension, lb per sq in	Ratio of modulus of elasticity of steel to that of concrete
New York, 1915...	650	500	40	150	80	16 000	16 000	{ 15 (girders)[a] 12 (columns)[a]
Chicago, 1917.....	35% ult.*	⅙ ult.*	....	1/18 ult.*	70 to 100	{ ⅓ E. L.† ≯18 000 }	{ shearing-stress 12 000 }	10 to 20 [c]
Philadelphia, 1918.	650	500	40	120	80 and 100	16 000	....	12 [b]
Boston, 1916......	500	{ 416 [a] 347 min.[d]	60	....	60	16 000	....	{ 15 (girders) 10 (columns)
Cleveland, 1918....	700	500	40	125	50 to 100	16 000	10 000	15
Baltimore, 1908...	500	400	50	....	....	15 000	....	15
Detroit..........	650	450	40	....	80 to 100	{ 45% E.L.† ≯16 000 }	....	{ 15 [a] 12 [b]
Buffalo..........	650	500	40	120	80 to 100	16 000	....	{ 15 [a] 12 [b]
San Francisco, 1912	{ ≯ 500 ¼ ult.*	{ ≯ 500 ⅕ ult.* }	75	....	60 to 100	{ 20 000 ⅓ E. L. }	....	15

* Ultimate strength. † Elastic limit. The symbol ≯ indicates "equal to or less than."

a, 1 : 2 : 4 mixture; b, 1 : 1½ : 3 mixture; c, according to mixture; d, in piers and columns.

el **Reinforcement.** The function of the steel reinforcement is to take up ngitudinal and diagonal tensile stresses and in some cases, as in columns . beams reinforced at the top, to give additional compressive strength.

d **or High Steel.** Two grades of steel are used for the reinforcement, STEEL and HIGH-CARBON STEEL. MILD OR MEDIUM STEEL is used for all ıral shapes and is the ordinary MERCHANT-STEEL. It has an ultimate strength of from 60 000 to 70 000 lb per sq in, and its elastic limit is about lf the ultimate strength. HIGH-CARBON STEEL has a greater percentage ›on and is therefore more brittle. Its ultimate strength is about 105 000 › elastic limit about 55 000 lb per sq in. The use of HIGH-CARBON STEEL permit greater stresses in the reinforcement, and consequently a less t of steel and a greater economy in construction. On account of its greater ıess, however, it is liable to sudden failures under stress. It is also often to be cracked or broken when sent to the work, and unless it is very care-ıspected there is great liability of defective material getting into the ıre. Furthermore, much of the so-called HIGH-CARBON STEEL has been ın practice, after testing, to fall far short of the specifications. Its use is re to be avoided, unless special care is taken to secure an absolutely re-ırticle and to have it inspected and tested. For large, important work uld be desirable. Ordinarily, however, mild steel should be used, as com-ly it is manufactured and sold under such standard conditions that it is . As the modulus of elasticity of high-carbon steel is practically the same of medium steel, the deformation under any given loading is the same re is no special advantage in the use of one over the other. Steel meeting cifications of the American Society for testing materials* for reinforcing-recommended. See Table III. The phosphorus in the steel should not ›.10% for Bessemer steel nor 0.05% for open-hearth steel. For slab and eam-reinforcement where wire or small rods are suitable, steel manu-l from Bessemer billets may be used with a TENSILE STRENGTH of 105 000, IELD-POINT of not less than 52 500 lb per sq in.

ding Stresses for Steel. The generally accepted WORKING STRESS for . steel is 16 000 lb per sq in in tension. Tests have shown that in cases he failure of reinforced-concrete beams is due to the failure of the rein-nt, the stress in the metal had not more than reached the YIELD-POINT. int is somewhat lower than the ELASTIC LIMIT. The working stress in l, therefore, should be a fixed proportion of the yield-point or the elastic t is held by some that this ratio should not be as high as one to two, e nearly one to three, reducing the working stress in mild steel as given › 10 000 or 12 000 lb per sq in. In using high-carbon steel they would : a similar ratio of the elastic limit, whatever that may be, according to ·dinarily 20 000 lb per sq in is taken as the working stress for high-car-l. Allowable WORKING STRESSES in steel reinforcement in various cities ı in Table II, page 912.

on-Members. Reinforcement is used in a variety of shapes and com-ı, nearly all of them patented and some of them forming the basis for SYSTEMS. Where the reinforcement is employed to take up tension, ıam or girder, the BOND between the concrete and the steel is relied upon p the TENSIONAL STRESSES in the steel. The plain bars depend entirely ADHESION of the steel and the concrete for the action of the two mate-›mbination, or the full tensile strength of the rod is developed by anchor-›ds into the concrete at the ends, in which case the beam becomes more

* American Society for Testing Materials Standards, 1918.

Manufacture	Billet-steel, Bessemer or open-hearth, rolled from new billets only							Rail-steel, rolled from standard tee rails	
Kind of bar	Plain			Deformed			Cold-twisted	Plain	Deformed and hot-twisted
Grade	Structural	Intermediate	Hard	Structural	Intermediate	Hard			
Tensile strength, pounds per square inch	55 000 to 70 000	70 000 to 85 000	80 000 or more	55 000 to 70 000	70 000 to 85 000	80 000 or more	....	80 000	80 000
Yield-point, minimum, pounds per square inch	33 000	40 000	50 000	33 000	40 000	50 000	55 000	50 000	50 000
Elongation, minimum, in 8 in, per cent	$\frac{1\,400\,000}{\text{tens. str.}}$	$\frac{1\,300\,000}{\text{tens. str.}}$	$\frac{1\,200\,000}{\text{tens. str.}}$	$\frac{1\,250\,000}{\text{tens. str.}}$	$\frac{1\,125\,000}{\text{tens. str.}}$	$\frac{1\,000\,000}{\text{tens. str.}}$	5	$\frac{1\,200\,000}{\text{tens. str.}}$	$\frac{1\,000\,000}{\text{tens. str.}}$
Cold-bending test around a pin, without fracture, under ¾ in	180° $d = t$	180° $d = 2t$	180° $d = 3t$	180° $d = t$	180° $d = 3t$	180° $d = 4t$	180° $d = 2t$	180° $d = 3t$	180° $d = 4t$
¾ in and over	180° $d = t$	90° $d = 2t$	90° $d = 3t$	180° $d = 2t$	90° $d = 3t$	90° $d = 4t$	180° $d = 3t$	90° $d = 3t$	90° $d = 4t$

NOTE. d is the diameter of the pin; t is the thickness or diameter of the bar.

ous to a trussed beam with the rod as the tension-member. In cross-
a, plain bars are usually round or square, though sometimes flat bars,
, tees, or other shapes are used. In regard to the use of square bars and
other shapes, it is contended that the edges start initial cracks in the con-
s it shrinks in setting. Twisted flat bars, when placed too near the sur-
the concrete, cause a spalling or breaking out of the concrete from between
nvolutions, when the steel is under stress.

mmercial Sizes. As a result of the shortage of steel during and since the
war, the larger producers of reinforcing-bars have agreed to eliminate
of the commercial sizes of bars formerly in use and are now limiting their
of bars to the following sizes:

Area	Equivalent to	Area	Equivalent to
0.110 sq in	3/8-in round	0.601 sq in	7/8-in round
0.196 sq in	1/2-in round	0.785 sq in	1-in round
0.250 sq in	1/2-in square	1.000 sq in	1-in square
0.307 sq in	5/8-in round	1.266 sq in	1 1/8-in square
0.442 sq in	3/4-in round	1.563 sq in	1 1/4-in square

ty in obtaining reinforcement will be avoided to a great extent by the
hese sizes in designing reinforced concrete.

rmed Bars. With the DEFORMED BARS the adhesion of the concrete to
el is supplemented by a MECHANICAL BOND due to the shape of the bar.
lowing deformed bars have been and are at present widely used.

Ransome Bar. The Ransome Twisted Bars (Fig. 1) are made of square
Bars should be "twisted cold with one complete twist in a length of not

Fig. 1. The Ransome Twisted Bar

elve times the thickness of the bar."* The work on the bars in the twist-
ess increases the elastic limit and the tensile strength; but the amount
ncrease is not fixed, as variations in the grade of rolled steel may result,
isting, in still wider variations. The users of this bar generally assume a
stress of 20 000 lb per sq in. The patent on this bar has expired and it
y be used by anyone. Strictly speaking, this is not a deformed bar.
rs can be obtained in all sizes, varying by 1/8 in from 3/8 to 1 1/4 in. Larger
o, can be obtained on special order.

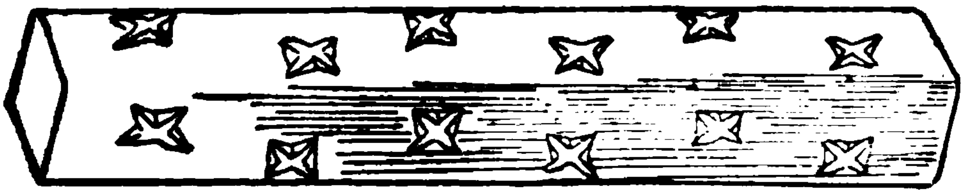


Fig. 2. The Buffalo Deformed Bar

uffalo Deformed Bar. The Buffalo Steel Company of Tonawanda,
akes a square bar with rounded edges, thus eliminating the sharp cor-

* ...an Society for Testing Materials Standards, 1918, pages 149 and 152.

ners. The deformations consist of raised stars along the sides of the bar, a shown in Fig. 2. It is made in sizes of from ⅜ to 1¼-in diameter, and th cross-sectional areas are equal to the areas of equivalent squares. The bars a rolled from old railroad rai and comply with the Standa Specifications of the Americ Society for Testing Material for reinforcing-steel of th kind. The steel is a hig carbon steel with a tensi strength of 80 000 lb per sq i

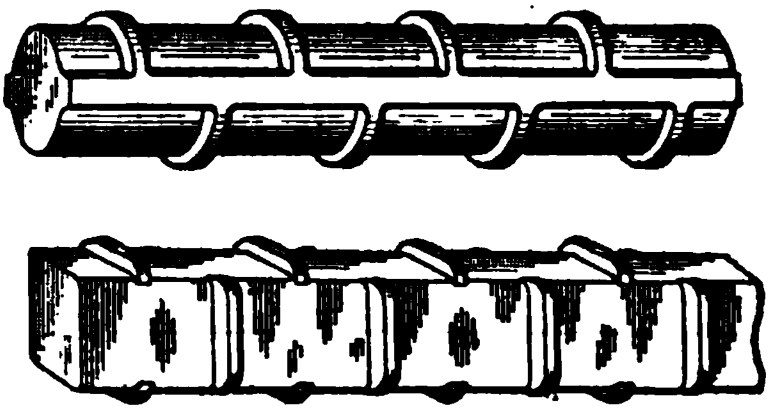


Fig. 3. Corrugated Bars. Round and Square

Corrugated Bars. Co rugated bars (Fig. 3), bot square and round in cros section, are made by the Co rugated Bar Company, Inc Buffalo, N. Y., of both medium and high-elastic-limit steel with a yield-point about 50 000 lb per sq in. Corr-Bars are furnished either straight and cut length, or bent ready for the forms. The standard sizes are as follows:

Corrugated Rounds

Size in inches	⅜	½	9/16	⅝	¾	⅞	1	1⅛	1¼
Net area in square inches	0.11	0.19	0.25	0.30	0.44	0.60	0.78	0.99	1.2
Weight per foot in pounds	0.38	0.66	0.86	1.05	1.52	2.06	2.69	3.41	4.2

Corrugated Squares

Size in inches	¼	⅜	½	⅝	¾	⅞	1	1⅛	1¼
Net area in square inches	0.06	0.14	0.25	0.39	0.56	0.76	1.00	1.26	1.5
Weight per foot in pounds	0.22	0.49	0.86	1.35	1.94	2.64	3.43	4.34	5.3

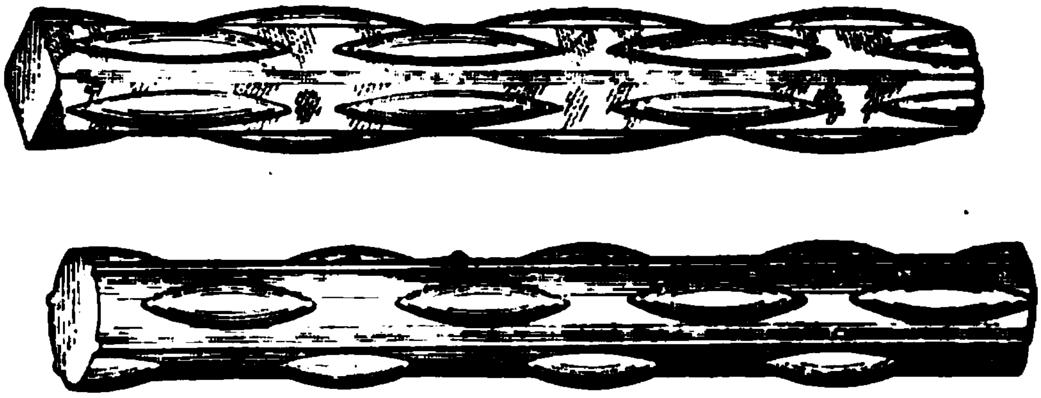


Fig. 4. The Havermeyer Bar, Square and Round

e **Havermeyer Bar.** The Havermeyer Bar (Fig. 4), controlled by the ete Steel Company, New York City, consists of square and round bars with a series of gradual projections and depressions on all sides, the defor-ns being so designed that there is a constant cross-sectional area. They rnished in the following sizes and weights:

in inches	Square bars		Round bars	
	Area in square inches	Weight per foot in pounds	Area in square inches	Weight per foot in pounds
¼	0.0625	0.212	0.0491	0.167
⅜	0.1406	0.478	0.1104	0.375
½	0.2500	0.850	0.1963	0.667
⅝	0.3906	1.328	0.3068	1.043
¾	0.5625	1.913	0.4418	1.502
⅞	0.7656	2.603	0.6013	2.044
1	1.0000	3.400	0.7854	2.670
1⅛	1.2656	4.303	0.9940	3.379
1¼	1 5625	5.312	1.2272	4.173
1⅜	1.8906	6.428	....	....
1½	2.2500	7.650	....	....

ation of 5% under and 2½% over the above weights is required for rolling.

company also rolls a flat bar with similar deformations on the wide faces. rm is recommended where bars must be bent in curves, as in silos, sewers, ı running them through a tire-machine to bend them, the edges of the event the lugs from being damaged.

Diamond Bar. The Diamond Bar (Fig. 5), put on the market by the e Steel Engineering Company, New York, is a bar of absolutely uniform

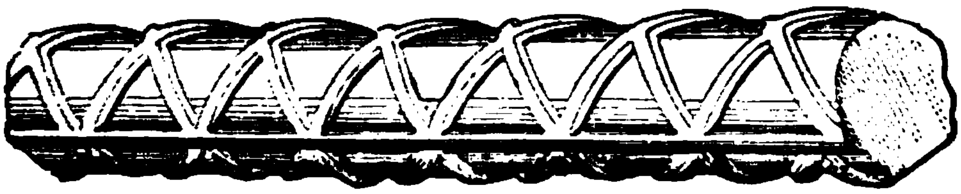


Fig. 5. The Diamond Bar

There is consequently no waste of metal due to the deformations. is practically a round bar, and as sudden transitions from one section to

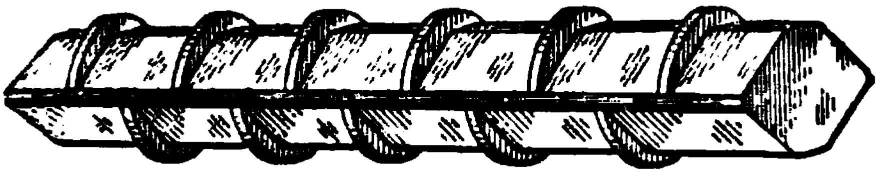


Fig. 6. The Rib-bar

ire avoided, all tendency to cause initial cracks in the concrete is over-he weights and areas of Diamond bars are equal to those of plain square ike denominations. Bars from ¼ to 1¼ in in diameter may be ob-

The Rib-Bar. The Rib-Bar (Fig. 6) manufactured by the Truscon Ste Company, Youngstown, Ohio, is a rolled bar with a series of cross-ribs. The bars are made with rectangular or round section and are furnished in sizes c from ⅜ to 1¼ in, the areas of the cross-sections being equivalent to squares c equal denominations; but the weights are slightly greater, and are as follows:

Size, in	Square bars		Round bars	
	Area, sq in	Weight per linear foot, lb	Area, sq in	Weight per linear foot, lb
⅜	0.1406	0.48	0.1104	0.379
½	0.2500	0.86	0.1963	0.674
⅝	0.3906	1.35	0.3068	1.054
¾	0.5625	1.95	0.4418	1.517
⅞	0.7656	2.65	0.6013	2.065
1	1.0000	3.46	0.7854	2.697
1⅛	1.2656	4.38	0.9940	3.414
1¼	1.5625	5.41	......	.....

Kalman Grip-Bars. These bars are similar in general design to the Ril Bars, differing from them by having the ribs running entirely around the bar instead of half-way. They are kept in stock in both round and square sectio of standard sizes and weights, by the Paul J. Kalman Company, of Chicago, Il

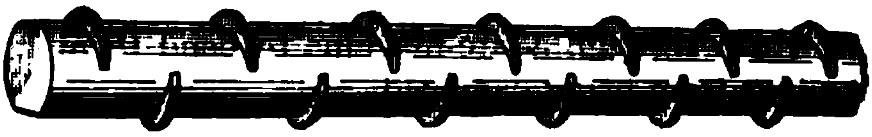


Fig. 7. The Ovoid Bar

The Ovoid Bar. The Gabriel Reinforcement Company, located at Detroi Mich., furnishes the Ovoid Bar (Fig. 7), in sizes and areas as follows:

Size in inches	⅜	½	⅝	¾	⅞	1	1⅛
Area in square inches...	0.1406	0.250	0.3906	0.5625	0.7656	1.000	1.26
Weight in pounds.......	0.4940	0.873	1.3560	1.9470	2.6430	3.446	4.35

The Monotype Bar. These bars are cruciform in section, and have, intervals, ribs connecting the stems. (Fig. 8.) The cross-sectional areas a

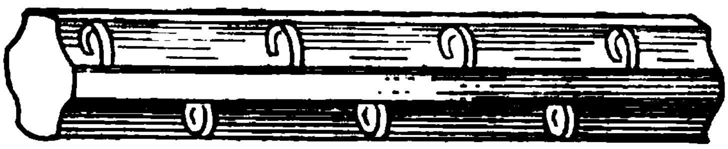


Fig. 8. The Monotype Bar

equivalent to those of standard round and square bars. They can be secu from the Edward A. Tucker Company, Boston, Mass.

Grip-Bars. These bars are rolled in sections equivalent to standard ars. The cross-section is especially designed so that shear-members

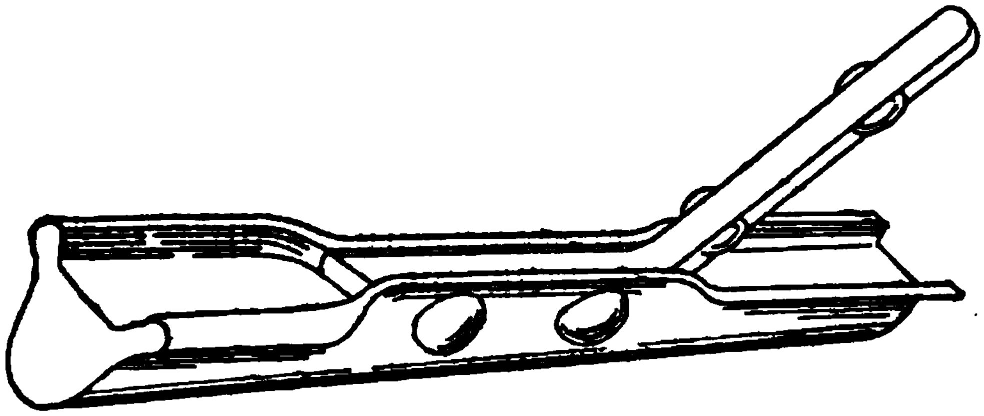


Fig. 9. The Rivet Grip-bar

gidly attached, as shown in Fig. 9, thus securing such advantages as are r them. They are handled by the Concrete Reinforcing and Engineer-any, Cleveland, Ohio.

Rivet grip-bars, size	Area, sq in	Perimeter, in	Weight, per foot, lb
⅜ in	0.1406	1.63	0.478
⅝ in	0.3906	4.00	1.328
¾ in	0.5625	4.25	1.913
⅞ in	0.7656	4.75	2.603
1 in	1.0000	5.19	3.400
1⅛ in	1.2656	5.75	4.303
1¼ in	1.5625	6.50	5.313

lesh and Expanded Metal. Other types of tension-reinforcement, RE-MESH FABRIC and EXPANDED METAL in various forms, have been n Chapter XXIII, Fireproofing of Buildings. Wire fabric has come general use as a slab-reinforcement, as it resists temperature-cracks cking of the concrete from impact or shock. It is made in various h heavy longitudinal or carrying wires and lighter transverse dis-r tie-wires. Expanded metal is similar to wire mesh in providing nt in both directions, rigidly spaced and attached or fastened to-is additional advantage is claimed for it; it provides reinforcement ions, thus taking care of concentrated loads.

ng. Different methods have been used for ANCHORING the tension-forced concrete. In the Hennebique system of construction (Fig. 10) bars are used, the ends of the rods are split and flared out. In other s the ends of the bars are simply turned at right-angles in such is most suitable. In some instances nuts and washers have been he ends of reinforcing-rods. Where reinforced-concrete floors are nection with steel columns the rods are run through the web-plates ngle-brackets and secured with nuts.

L. The strengths of the BOND between concrete and steel for various rs and differing conditions are shown in Table IV. After the bond he reinforcement still acts in conjunction with the concrete, due to or FRICTIONAL RESISTANCE. Numerous tests have shown this fric-

tional resistance to be about two thirds of the initial bond-strength. BOND-STRENGTH for ordinary round or square-section bars may be taken at

Fig. 10. The Hennebique System

to 300 lb per sq in, depending upon the character of the concrete and the de of roughness of the steel. MECHANICAL BOND depends upon the shape of bar and the compressive and shearing strength of the concrete.

Table IV. Results of Tests on Adhesion Between Concrete and Steel

Kind of bar	Size tested in fraction of inch	Concrete	Age	Ultimate stre developed in l per sq in of su face in conta
Round	½	1:2:4	60 days	412 (a)
Square	¾	1:3:6	30 days	274 (b)
Square (rusted)	¾	....	30 days	437 (c)
Square (rusted)	¾	....	90 days	642 (c)
Square	⅞	....	90 days	431 (c)
Square	⅞	....	30 days	294 (c)
Twisted (Ransome)	⅝	1:2:4	31 days	648 (d)
Twisted	¾	....	25 days	500 (c)
Twisted	¾	Neat cement	7 mos.	1 290 (e)
Twisted	¾	1:1	7 mos.	1 318 (e)
Twisted	¾	1:2	7 mos.	1 199 (e)
Twisted	¾	1:3	7 mos.	701 (e)
Twisted	¾	1:4	7 mos.	796 (e)
Corrugated	¾	Neat cement	7 mos.	962 (e)
Corrugated	¾	1:1	7 mos.	977 (e)
Corrugated	¾	1:2	7 mos.	934 (e)
Corrugated	¾	1:3	7 mos.	735 (e)
Corrugated	¾	1:4	7 mos.	564 (e)
Corrugated	¾	1:2:4	31 days	640 (d)
Thatcher (f)	⅞	....	30 days	646 (c)

The following are the authorities for the above tests:

(a) A. N. Talbot. (b) C. M. Spofford. (c) New York City Rapid Transi
(d) T. L. Condron. (e) Tests of Metals, Watertown Arsenal, 1904.
(f) No longer manufactured.

:-**Members.** In many of the tests on full-sized concrete beams, failure y the development of diagonal breaks near the supports. The first crack in a beam, with nothing but horizontal tension-steel at the is apt to occur when the maximum VERTICAL SHEAR is from 100 to 200 lb . Since the vertical shear is accompanied by a HORIZONTAL SHEAR of ensity in all parts of the beam, it was formerly thought that this diag- ire was due to these shearing-forces at the end of the beam and vertical or bent-up rods were provided to resist the horizontal shear. More sts have shown that the SHEARING STRENGTH of concrete is from 60 to he compressive strength, and that these cracks are diagonal and in the which could be expected from the THEORY OF DIAGONAL TENSION, tributes them to a combination of the shearing-stress with the hori- isile stress. The inclined cracks which first appear are due to a rupture acrete in tension. The most effective way to prevent this rupture is e reinforcement in the direction of the stress that is inclined upwards ie supports, as nearly as possible normal to the line of the diagonal 'ertical reinforcement could be used, but it would not act until def- or downward displacement of the concrete occurred on the side of the y from the support. If vertical stirrups are used for this reinforce- y must be spaced a less distance apart than the effective depth of the l they must be looped around, though not necessarily attached to, the bars. When inclined reinforcement is used, it must be rigidly at- the longitudinal members and spaced a less distance apart than the epth of the beam. The reason for this is that the magnitude and in- f the diagonal tension increases from the middle toward the end of the g inclined 45° where the horizontal tension becomes zero.

ihn **Bar.** In the Kahn Trussed Bar (Fig. 11) the attachment of the the tension-member is positively secured. The bars are square or

Fig. 11. The Kahn Bar

in cross-section with webs rolled on them at two diagonally opposite e stirrups are formed by shearing these webs through a part of their turning up parts, as shown in the cut. These stirrups may be placed n up in pairs or so as to alternate on opposite sides of the bar, making of the stirrups closer than when turned up in pairs. Another advan- ital to the use of this bar is that the greater effective cross-section in at the middle, the point of greatest bending moment with the usual 'wo disadvantages, however, are the separation of the concrete by bove and below the bar, and the limitation as to the effective stirrup- eep beams. This bar is controlled by the Truscon Steel Company, 1, Ohio.

1 Trussed Bar can be obtained in the sizes shown in table on page

Compression. The steel reinforcement in reinforced concrete is ain cases to assist in developing COMPRESSIVE STRENGTH when the iot sufficient for the purpose, as in the case of beams and girders with

Size, in	Weight per linear foot, lb	Area, sq in	Standard le[ngths] of diago[nals] in
½ × 1½	1.4	0.41	12
¾ × 2 5/16	2.7	0.79	12–24
1½ × 2¾	4.8	1.41	12–24–36
1⅛ × 2¾	6.8	2.00	36
2 × 3½	10.2	3.00	36

rods placed above the neutral axis, and columns with rods placed verti The use of the steel reinforcement in resisting COMPRESSION will be treated at length in Subdivision 3 of this chapter, in the paragraph Compression in Beams and Girders, page 941. On account of the uncertainty, howe the steel and concrete each receiving its proportionate share of the load, t of steel in compression should be avoided as much as possible.

The Position of the Reinforcement. The importance of the EXACT TION OF THE REINFORCEMENT in the concrete will become more apparent discussion of the design of beams. A slight displacement of the steel wi terially affect the strength. If the steel shifts upward the beam is weake it shifts downward the protection of the steel against rust or fire is reduce the so-called UNIT SYSTEMS the reinforcements, including the tension-ro stirrups, are so tied and framed together that after being placed in the the possibility of shifting their positions with respect to the other surfa the beam or to one another is practically entirely removed.

The Unit System. The particular advantages in the use of a UNIT SY of reinforcement is, as already indicated, the assurance that each and every

Fig. 12. The Unit System

of the reinforcement is in its exact relative position, and maintains that po during the placing of the concrete. The reinforcement for each beam or g is as carefully laid out as the location of cover-plates, stiffeners, connection-a and rivets in a built-up steel girder. It can consequently be thoroughl spected and checked before being placed in position. Being marked, its location is easily determined by the foreman on the job, from the ere plan After it is put in place a quick inspection will show at once wheth correctly placed or not, as it must fit and extend the full length of the Being fabricated OFF THE JOB there is less interference between workmen.

on can proceed while the molds are being made, and consequently peed in erection is possible. The frames are readily transported and : to get mixed than loose rods sent to the job.

nit System (Fig. 12) is the pioneer of this type of construction and ıctured by the American System of Reinforcing, Chicago, Ill. Its par- atures are the bending up of some of the longitudinal reinforcements ıupports and the use of round U-shaped stirrups, wound around the bars and allowed to shrink into place.

ummings System (Fig. 13) is manufactured by the Electric Welding ', Pittsburgh, Pa. The particular feature of this system is the forming

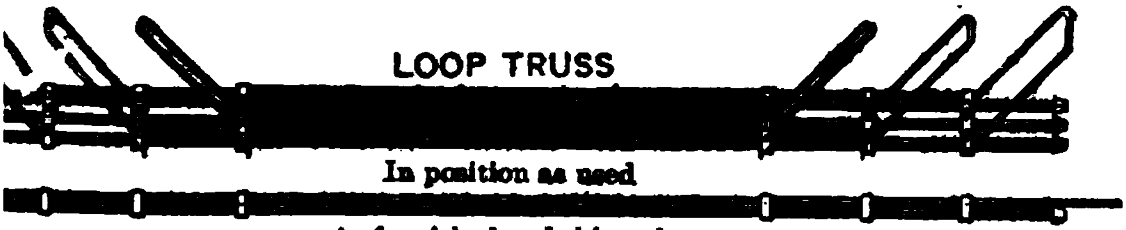


Fig. 13. The Cummings System

layer of small rods into rectangular frames which, after being fastened r layer at suitable points, permit the bending up of the ends to act as bers or stirrups, thus utilizing for shear the steel that is not required ; moments.

ten Truss. The Luten Truss (Fig. 14) consists of longitudinal rods ıate members bent diagonally upwards across the beam and con-

Fig. 14. The Luten Truss

ıg the upper surface to the end of the beam. Diagonal members ' through all the region of diagonal tension in both ends of the beam. s provided with a clamp and wedge that locks the members together. ed by the National Concrete Company, Indianapolis, Ind.

-Bar Unit. The Corr-Bar Unit, Fig. 15, made by the Corrugated y, Inc., Buffalo, N. Y., is provided with a continuous stirrup of both

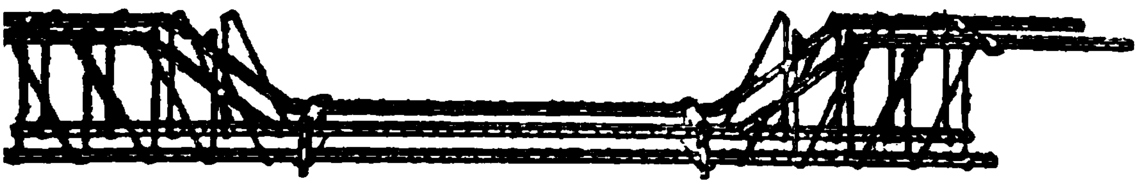


Fig. 15. The Corr-bar Unit

nclined web-members with a rigid anchorage at both top and bot- r by Professor Talbot on this type of reinforced beam, considerably than ordinary were obtained in vertical shear.

3. Design of Reinforced-Concrete Construction

Girders, Beams, and Slabs. Different formulas for the design of reinfo concrete girders, beams, slabs, etc., based on various theoretical considera have been devised by different investigators. The formulas here given have widely accepted and are offered because they are simple in form and give sat tory results. If anything, they err on the side of safety; and furthermore, have been found to give results closely in accord with actual tests. They are by the New York City Building Bureau, and are accepted by other authoriti

Assumptions in the Formulas. The formulas are based on the foll assumptions:

(1) The BOND between the concrete and steel is sufficient to make th materials act together.

(2) A PLANE CROSS-SECTION of a beam before bending remains a plane se after bending, and the stress and strain* in any fiber of either materia directly proportional to the distance of that fiber from the neutral axis c cross-section.

(3) The MODULUS OF ELASTICITY of the concrete in compression remains stant within the assumed working stresses.

(4) The TENSIONAL STRESS is taken entirely by the steel; that is, the t strength of the concrete is not considered.

Fig. 16 represents a longitudinal section and a cross-section of a reinfo concrete beam in a state of flexure or bending under a load. The fibers a

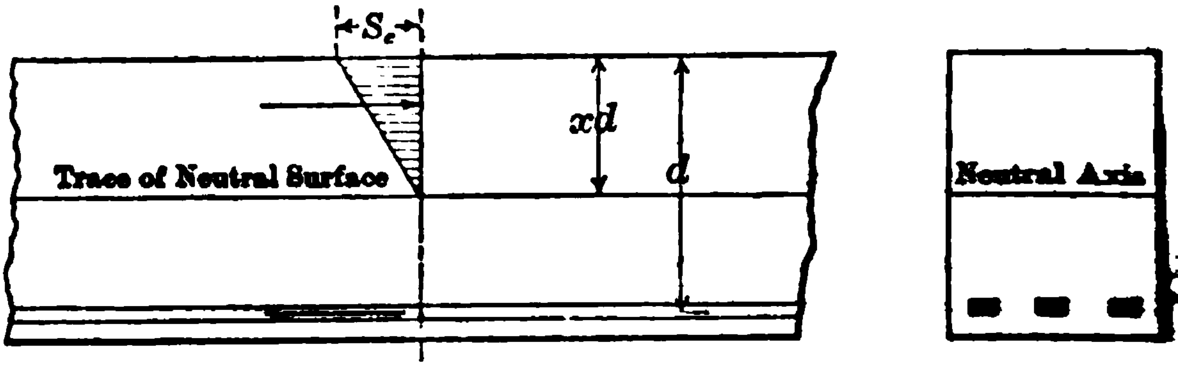


Fig. 16. Sections of Reinforced-concrete Beam

the NEUTRAL SURFACE of the beam or above the NEUTRAL AXIS of the cro tion are in compression and according to the assumptions the stresses v direct proportion to their distances from the neutral surface or axis, so th total area of compression in the concrete, representing the TOTAL COMPRE STRESS, may be graphically indicated by the shaded triangle. The TOTAL SIONAL STRESS may be assumed to be concentrated at the center of gravity cross-section of the steel reinforcement. One of the conditions of STATIC LIBRIUM for the beam is that the algebraic sum of all the horizontal stre the cross-section shall be zero; that is, that the sum of all the compr stresses, or the resultant compressive stress in the concrete, must equal th or resultant tensional stress in the steel.

Formulas for Reinforced-Concrete Beams. From these assump based upon THEORETIC and EXPERIMENTAL LAWS, the following formul derived, in which

S_t = the allowable unit tension or working stress in the steel in poun square inch;

* Deformation.

ı the allowable unit compression or working stress in the extreme fibers of the concrete in pounds per square inch;
the ratio of the modulus of elasticity of the steel to the modulus of elasticity of the çoncrete;
the effective depth of the beam, in inches; the distance from center of gravity of steel reinforcement to extreme fibers in compression;
the ratio of the depth of the neutral axis from the extreme fibers in compression, to the effective depth of the beam, so that
the distance of the neutral axis, in inches, from the extreme fibers in compression;
he width of the beam in inches;
the ratio of the cross-section of the steel to the cross-section of the beam, considering the beam all of that part of the concrete above the center of gravity of the steel;
the maximum bending moment at the dangerous section of the beam in inch-pounds;
:he moment of resistance, in inch-pounds, at the dangerous section of the beam, and must of course be equal to or potentially greater than the maximum bending moment;*
factor used for simplification of the formulas. This factor is constant for any given steel and concrete;
ectional area of the steel in square inches.
ns of rectangular cross-section

$$M = M_r = Kbd^2 \qquad (1)$$

f K being determined by the formula

$$K = S_t \left(\frac{1}{2\left(\frac{S_t}{S_c}\right)\left(1+\frac{S_t}{S_{c}r}\right)}\right)\left(1 - \frac{1}{3\left(1+\frac{S_t}{S_{c}r}\right)}\right) \qquad (2)$$

ıla can be deduced from the LAWS OF FLEXURE of beams and the ; noted above.
: of this formula for the value of K it must be remembered that the o S_c for any given ratio of steel to concrete, p, is a constant, so that ıg values of S_t and S_c must be used. This ratio, p, often spoken of :NTAGE OF REINFORCEMENT, is the expression in the first parenthesis d member of Formula (2)

$$p = \frac{1}{2\left(\frac{S_t}{S_c}\right)\left(1+\frac{S_t}{S_{c}r}\right)} \qquad (3)$$

ıent of resistance" or the "resisting moment" referred to any cross-section horizontal position and in a state of flexure under a load or loads is the ıf the moments of the internal horizontal stresses with reference to a point and the "bending moment" for that section is the algebraic sum of the the external vertical forces on either side of the section with reference oint (the forces on the left side being usually taken). The resisting ıse with the bending moments and in the flexure formula, $M = SI/c$ (see ıd X), they are made equal to each other, M being the bending moment resisting moment. In the following formulas, M and the expression :nt" generally, denote the maximum bending moment. M_{max} is often the latter.

The value of x is derived from the expression

$$x = rp\left(\sqrt{1+\frac{2}{rp}}-1\right)$$

Values for K and x for corresponding values of p, for different conditi fixed by the building authorities of different cities, are given in Tables V, VII and VIII.

Table V. Values for Formulas for Reinforced Concrete

$r = 12$

p	x	K	S_e	S_t	K	S_e	S_t	K	S_e	S_t
0.0045	0.279	.....	...	......	....	...	......	65.4	516	16
0.0050	0.291	.....	...	......	....	...	......	72.2	550	"
0.0055	0.303	.....	...	......	69.3	507	14 000	79.2	580	"
0.0058	0.310	.....	...	......	73.0	525	"	83.4	600	"
0.0060	0.314	.....	...	......	75.2	535	"	86.0	612	"
0.0065	0.325	.....	...	......	81.2	560	"	92.8	640	"
0.0070	0.334	74.7	503	12 000	87.2	587	"	96.5	650	15
0.0075	0.344	79.5	523	"	93.0	610	"	99.0	"	14
0.0080	0.353	84.7	544	"	98.8	635	"	101.2	"	14
0.0085	0.361	89.9	565	"	103.3	650	13 800	103.3	"	13
0.0090	0.369	94.7	584	"	105.1	"	13 350	105.1	"	13
0.0095	0.377	99.6	605	"	107.0	"	12 900	107.0	"	12
0.0100	0.384	104.5	625	"	108.8	"	12 500	108.8	"	12
0.0105	0.392	109.5	643	"	110.9	"	12 130	110.9	"	12
0.0110	0.399	112.4	650	11 790	112.4	"	11 790	112.4	"	11
0.0115	0.405	114.0	"	11 450	114.0	"	11 450	114.0	"	11
0.0120	0.412	115.7	"	11 160	115.7	"	11 160	115.7	"	11
0.0125	0.418	117.2	"	10 900	117.2	"	10 900	117.2	"	10
0.0130	0.424	118.2	"	10 600	118.2	"	10 600	118.2	"	10
0.0135	0.430	119.8	"	10 350	119.8	"	10 350	119.8	"	10
0.0140	0.436	121.2	"	10 120	121.2	"	10 100	121.2	"	10
0.0145	0.441	122.2	"	9 890	122.2	"	9 870	122.2	"	9
0.0150	0.446	123.2	"	9 660	123.2	"	9 660	123.2	"	9
0.0155	0.452	124.8	"	9 460	124.8	"	9 460	124.8	"	9
0.0160	0.457	126.0	"	9 270	126.0	"	9 270	126.0	"	9
0.0165	0.462	127.0	"	9 100	127.0	"	9 100	127.0	"	9
0.0170	0.467	128.0	"	8 930	128.0	"	8 930	128.0	"	8
0.0175	0.471	129.1	"	8 740	129.1	"	8 740	129.1	"	8
0.0180	0.475	130.1	"	8 580	130.1	"	8 580	130.1	"	8
0.0185	0.480	131.0	"	8 440	131.0	"	8 440	131.0	"	8
0.0190	0.485	132.1	"	8 300	132.1	"	8 300	132.1	"	8
0.0195	0.489	133.0	"	8 150	133.0	"	8 150	133.0	"	8
0.0200	0.493	134.0	"	8 010	134.0	"	8 010	134.0	"	8

Table VI. Values for Formulas for Reinforced Concrete

$r = 12$

x	K	S_c	S_t	K	S_c	S_t	K	S_c	S_t
0.217	.....	...	......	....	...	......	51.0	506	22 000
0.235	.....	...	......	55.3	511	20 000	60.8	562	"
0.251	57.7	503	18 000	64.2	558	"	70.6	614	"
0.266	65.7	542	"	72.9	602	"	78.7	650	21 610
0.279	73.5	581	"	81.6	645	"	82.3	"	20 150
0.291	81.3	618	"	85.4	650	18 910	85.4	"	18 910
0.303	88.5	650	17 900	88.5	"	17 900	88.5	"	17 900
0.314	91.5	"	17 000	91.5	"	17 000	91.5	"	17 000
0.325	94.2	"	16 250	94.2	"	16 250	94.2	"	16 250
0.334	96.5	"	15 510	96.5	"	15 510	96.5	"	15 510
0.344	99.0	"	14 900	99.0	"	14 900	99.0	"	14 900
0.353	101.2	"	14 350	101.2	"	14 350	101.2	"	14 350
0.361	103.3	"	13 800	103.3	"	13 800	103.3	"	13 800
0.369	105.1	"	13 325	105.1	"	13 325	105.1	"	13 325
0.377	107.0	"	12 900	107.0	"	12 900	107.0	"	12 900
0.384	108.8	"	12 480	108.8	"	12 480	108.8	"	12 480
0.392	110.9	"	12 130	110.9	"	12 130	110.9	"	12 130
0.399	112.4	"	11 790	112.4	"	11 790	112.4	"	11 790
0.405	113.9	"	11 450	113.9	"	11 450	113.9	"	11 450
0.412	115.6	"	11 160	115.6	"	11 160	115.6	"	11 160
0.418	117.2	"	10 900	117.2	"	10 900	117.2	"	10 900
0.424	118.4	"	10 600	118.4	"	10 600	118.4	"	10 600
0.430	120.0	"	10 350	120.0	"	10 350	120.0	"	10 350
0.436	121.2	"	10 100	121.2	"	10 100	121.2	"	10 100
0.441	122.2	"	9 870	122.2	"	9 870	122.2	"	9 870
0.446	123.2	"	9 660	123.2	"	9 660	123.2	"	9 660
0.452	124.8	"	9 460	124.8	"	9 460	124.8	"	9 460
0.457	126.0	"	9 270	126.0	"	9 270	126.0	"	9 270
0.462	127.0	"	9 100	127.0	"	9 100	127.0	"	9 100
0.467	128.0	"	8 930	128.0	"	8 930	128.0	"	8 930
0.471	129.1	"	8 740	129.1	"	8 740	129.1	"	8 740
0.475	130.1	"	8 580	130.1	"	8 580	130.1	"	8 580
0.480	131.0	"	8 440	131.0	"	8 440	131.0	"	8 440
0.485	132.1	"	8 300	132.1	"	8 300	132.1	"	8 300
0.489	133.0	"	8 150	133.0	"	8 150	133.0	"	8 150
0.493	134.0	"	8 010	134.0	"	8 010	134.0	"	8 010

Table VII. Values for Formulas for Reinforced Concrete

$r = 15$

p	z	K	S_c	S_t	K	S_c	S_t	K	S_c	S_t
0.0050	0.320	.....	...	......	....	...	......	71.6	500	16 00
0.0055	0.332	.....	...	......	....	...	......	78.3	530	"
0.0060	0.344	.....	...	......	....	...	......	85.1	558	"
0.0065	0.355	.....	...	......	80.2	513	14 000	91.6	586	"
0.0070	0.365	.....	...	......	86.1	537	"	98.3	614	"
0.0075	0.375	.....	...	......	92.0	560	"	105.1	640	"
0.0080	0.384	83.6	500	12 000	97.6	583	"	108.9	650	15 60
0.0085	0.393	88.6	519	"	103.3	606	"	111.0	"	15 0
0.0090	0.402	93.5	537	"	109.0	627	"	113.2	"	14 52
0.0095	0.410	98.4	556	"	114.8	648	"	115.1	"	14 02
0.0100	0.418	103.3	573	"	117.1	650	13 600	117.1	"	13 60
0.0105	0.425	108.2	593	"	118.6	"	13 150	118.6	"	13 15
0.0110	0.433	113.1	611	"	120.5	"	12 760	120.5	"	12 76
0.0115	0.440	117.9	627	"	122.0	"	12 420	122.0	"	12 42
0.0120	0.446	122.7	647	"	123.4	"	12 080	123.4	"	12 08
0.0125	0.453	125.0	650	11 780	125.0	"	11 780	125.0	"	11 78
0.0130	0.459	126.8	"	11 480	126.8	"	11 480	126.8	"	11 48
0.0135	0.465	127.7	"	11 200	127.7	"	11 200	127.7	"	11 20
0.0140	0.471	128.9	"	10 920	128.9	"	10 920	128.9	"	10 92
0.0145	0.477	130.4	"	10 690	130.4	"	10 690	130.4	"	10 69
0.0150	0.483	131.7	"	10 465	131.7	"	10 465	131.7	"	10 46
0.0155	0.488	133.0	"	10 240	133.0	"	10 240	133.0	"	10 24
0.0160	0.493	133.9	"	10 010	133.9	"	10 010	133.9	"	10 01
0.0165	0.498	135.2	"	9 810	135.2	"	9 810	135.2	"	9 81
0.0170	0.503	136.0	"	9 620	136.0	"	9 620	136.0	"	9 62
0.0175	0.508	137.2	"	9 435	137.2	"	9 435	137.2	"	9 43
0.0180	0.513	138.2	"	9 260	138.2	"	9 260	138.2	"	9 26
0.0185	0.518	139.4	"	9 100	139.4	"	9 100	139.4	"	9 10
0.0190	0.522	140.3	"	8 940	140.3	"	8 940	140.3	"	8 94
0.0195	0.527	141.1	"	8 790	141.1	"	8 790	141.1	"	8 79
0.0200	0.531	142.0	"	8 630	142.0	"	8 630	142.0	"	8 63

Table VIII. Values for Formulas for Reinforced Concrete

$r = 15$

x	K	S_c	S_t	K	S_c	S_t	K	S_c	S_t
0.258	.....	...	......	....	...	......	60.3	512	22 000
0.276	.....	...	......	63.5	507	20 000	69.9	557	"
0.292	.....	...	......	72.3	548	"	79.5	604	"
0.306	72.7	528	18 000	80.7	587	"	88.8	646	"
0.320	80.5	563	"	89.4	626	"	92.9	650	20 800
0.332	88.1	596	"	96.0	650	19 610	96.0	"	19 610
0.344	95.6	628	"	99.1	"	18 620	99.1	"	18 620
.355	101.8	650	17 760	101.8	"	17 760	101.8	"	17 760
.365	104.1	"	16 950	104.1	"	16 950	104.1	"	16 950
.375	106.7	"	16 250	106.7	"	16 250	106.7	"	16 250
.384	108.9	"	15 600	108.9	"	15 600	108.9	"	15 600
.393	111.0	"	15 040	111.0	"	15 040	111.0	"	15 040
402	113.2	"	14 520	113.2	"	14 520	113.2	"	14 520
410	115.1	"	14 020	115.1	"	14 020	115.1	"	14 020
418	117.1	"	13 600	117.1	"	13 600	117.1	"	13 600
425	118.6	"	13 150	118.6	"	13 150	118.6	"	13 150
433	120.5	"	12 760	120.5	"	12 760	120.5	"	12 760
40	122.0	"	12 420	122.0	"	12 420	122.0	"	12 420
46	123.4	"	12 080	123.4	"	12 080	123.4	"	12 080
53	125.0	"	11 780	125.0	"	11 780	125.0	"	11 780
9	126.4	"	11 480	126.4	"	11 480	126.4	"	11 480
5	127.7	"	11 200	127.7	"	11 200	127.7	"	11 200
1	128.9	"	10 920	128.9	"	10 920	128.9	"	10 920
7	130.4	"	10 690	130.4	"	10 690	130.4	"	10 690
	131.7	"	10 465	131.7	"	10 465	131.7	"	10 465
	133.0	"	10 240	133.0	"	10 240	133.0	"	10 240
	133.9	"	10 010	133.9	"	10 010	133.9	"	10 010
	135.0	"	9 810	135.0	"	9 810	135.0	"	9 810
	136.0	"	9 620	136.0	"	9 620	136.0	"	9 620
	137.2	"	9 435	137.2	"	9 435	137.2	"	9 435
	138.2	"	9 260	138.2	"	9 260	138.2	"	9 260
	139.4	"	9 100	139.4	"	9 100	139.4	"	9 100
	140.1	"	8 940	140.1	"	8 940	140.1	"	8 940
	141.1	"	8 790	141.1	"	8 790	141.1	"	8 790
	142.0	"	8 630	142.0	"	8 630	142.0	"	8 630

Cinder Concrete. Values of K for cinder concrete are given in Tables I and X, which are, however, recommended to be used only for slabs. Cind concrete, though an excellent fireproofing material, lacks strength and shoul be used as a structural material for the slabs, only, between the beams.

Table IX. Values for Formulas for Reinforced Cinder Concrete

$r = 35$

p	x	K	S_c	S_t	K	S_c	S_t
0.0005	0.170	7.5	94	16 000	7.5	94	16 000
0.0010	0.232	14.8	138	"	14.8	138	16 000
0.0015	0.276	21.8	174	"	18.8	150	13 800
0.0020	0.311	28.7	206	"	20.9	"	11 633
0.0025	0.340	33.9	225	15 300	22.6	"	10 200
0.0030	0.365	36.1	"	13 688	24.0	"	9 125
0.0035	0.387	37.9	"	12 439	25.3	"	8 293
0.0040	0.407	39.6	"	11 447	26.4	"	7 631
0.0045	0.425	41.0	"	10 625	27.4	"	7 083
0.0050	0.442	42.4	"	9 945	28.3	"	6 630
0.0055	0.457	43.6	"	9 348	29.1	"	6 232
0.0060	0.471	44.7	"	8 831	29.8	"	5 888
0.0065	0.484	45.7	"	8 377	30.4	"	5 585
0.0070	0.497	46.7	"	7 988	31.1	"	5 325
0.0075	0.508	47.5	"	7 620	31.6	"	5 080
0.0080	0.519	48.3	"	7 298	32.2	"	4 866
0.0085	0.529	49.0	"	7 001	32.7	"	4 668
0.0090	0.539	49.7	"	6 738	33.2	"	4 492
0.0095	0.548	50.4	"	6 489	33.6	"	4 326
0.0100	0.557	51.0	"	6 266	34.0	"	4 178
0.0105	0.565	51.6	"	6 054	34.4	"	4 036
0.0110	0.573	52.1	"	5 860	34.8	"	3 907
0.0115	0.581	52.7	"	5 684	35.1	"	3 789
0.0120	0.588	53.2	"	5 513	35.5	"	3 675
0.0125	0.595	53.7	"	5 355	35.8	"	3 570
0.0130	0.602	54.1	"	5 210	36.1	"	3 473
0.0135	0.608	54.5	"	5 067	36.4	"	3 378
0.0140	0.615	55.0	"	4 942	36.7	"	3 295
0.0145	0.621	55.4	"	4 818	36.9	"	3 212
0.0150	0.626	55.7	"	4 695	37.1	"	3 130
0.0155	0.632	56.1	"	4 587	37.4	"	3 058
0.0160	0.637	56.4	"	4 479	37.6	"	2 986
0.0165	0.643	56.8	"	4 384	37.9	"	2 923
0.0170	0.648	57.2	"	4 288	38.1	"	2 859
0.0175	0.652	57.4	"	4 191	38.3	"	2 794
0.0180	0.657	57.7	"	4 106	38.5	"	2 738
0.0185	0.662	58.1	"	4 026	38.7	"	2 684
0.0190	0.666	58.3	"	3 943	38.9	"	2 629
0.0195	0.671	58.6	"	3 871	39.1	"	2 581
0.0200	0.675	58.9	"	3 797	39.2	"	2 531

Table X. Values for Formulas for Reinforced Cinder Concret

$r = 30$

x	K	S_c	S	K	S_c	S	K	S_c
0.159	7.6	100.6	16 000	7.6	100.6	16 000	6.62	88
0.216	14.9	148	"	14.9	148	"	13.0	129.
0.259	22.0	185	"	22.0	185	"	19.2	162
0.292	28.8	219	"	28.8	219	"	25.2	192
0.319	35.8	251	"	35.6	250	15 950	28.5	200
0.344	42.6	279	"	38.1	"	14 300	30.4	"
.365	48.1	300	15 620	40.1	"	13 030	32.0	"
.386	50.4	"	14 480	42.1	"	12 060	33.6	"
.402	52.2	"	13 400	43.4	"	11 170	34.8	"
.418	54.0	"	12 540	45.0	"	10 450	36.0	"
.433	55.6	"	11 810	46.3	"	9 860	37.0	"
.447	57.0	"	11 180	47.5	"	9 320	38.0	"
.460	58.5	"	10 620	48.7	"	8 850	38.9	"
.472	59.7	"	10 120	49.7	"	8 440	39.8	"
483	60.7	"	9 660	50.6	"	8 050	40.5	"
494	61.9	"	9 270	51.6	"	7 730	41.3	"
504	63.0	"	8 900	52.5	"	7 420	42.0	"
514	63.9	"	8 560	53.3	"	7 130	42.6	"
523	64.9	"	8 250	54.1	"	6 870	43.2	"
532	65.7	"	7 980	54.7	"	6 650	43.7	"
540	66.4	"	7 710	55.4	"	6 420	44.3	"
47	67.2	"	7 460	55.9	"	6 220	44.7	"
55	67.8	"	7 240	56.5	"	6 040	45.2	"
62	68.5	"	7 020	57.1	"	5 850	45.7	"
69	69.3	"	6 830	57.7	"	5 680	46.2	"
76	69.8	"	6 650	58.2	"	5 540	46.5	"
2	70.4	"	6 460	58.6	"	5 380	46.8	"
8	71.0	"	6 310	59.2	"	5 260	47.3	"
4	71.5	"	6 140	59.5	"	5 120	47.6	"
)	72.0	"	6 000	60.0	"	5 000	48.0	"
5	72.6	"	5 860	60.5	"	4 880	48.4	"
	73.1	"	5 730	60.9	"	4 780	48.7	"
	73.6	"	5 610	61.3	"	4 670	49.0	"
	74.0	"	5 480	61.7	"	4 570	49.4	"
	74.5	"	5 370	62.0	"	4 470	49.6	"
	74.9	"	5 270	62.4	"	4 390	49.9	"
	75.3	"	5 160	62.7	"	4 300	50.2	"
	75.7	"	5 060	63.1	"	4 220	50.4	"
	76.0	"	4 960	63.3	"	4 130	50.7	"
	76.4	"	4 870	63.6	"	4 060	50.8	"

Concrete Beams of Rectangular Cross-Section. In t BEAM required for any given case, r and the limiting nerally given, and K can be determined for any ratio, The value of M, the MAXIMUM BENDING MOMENT, th at the DANGEROUS SECTION of the beam, is determi loading, the span and the spacing; and the width a to be found. Formula (1) may then be put in the n

$$d = \sqrt{\frac{M}{Kb}}$$

A value for b is assumed and the equation solved for d. Architectural structural reasons will often limit the width or depth and several trials may ha to be made.

Reinforced-Concrete Slabs. For the STRENGTH OF SLABS the same formul apply. A slab may be treated (1) as a rectangular beam of unusual widt (2) as a series of beams set one alongside the other, the width of each beam bei equal to the spacing of the reinforcing-rods, and one rod being used for ea beam; or (3) as a series of beams of unit width, the area of steel for each bea being the area of reinforcement per unit of width.

Check-Formulas. It may sometimes happen that it is advisable to che a given or existing beam-construction as to strength or compliance with spe fications for working stresses. In that case the following formulas will l convenient (see, also, page 992):

$$M = pS_tbd^2\left(1 - \frac{x}{3}\right) \quad (6)$$

$$M = \frac{S_cxbd^2}{2}\left(1 - \frac{x}{3}\right) \quad (7)$$

If the strength of the beam for the assumed working stresses is to be dete mined, these values of S_t and S_c are inserted in Formulas (6) and (7), and th least value of M is used. If the values of M resulting from these equations a not equal, the full benefit of one of the materials is not being obtained. If th stresses in the steel or concrete due to a given loading are to be determined th formulas are put in the following forms:

$$S_t = \frac{M}{pbd^2\left(1 - \frac{x}{3}\right)} \quad (8)$$

$$S_c = \frac{2M}{xbd^2\left(1 - \frac{x}{3}\right)} \quad (9)$$

These formulas apply to rectangular beams only. M in Formulas (8) an (9) is the maximum moment due to the external forces, or the maximum ben ing moment. The value of x can be determined from Tables V to X. In Fo mula (8) it will be noted that the denominator of the fraction is an expressio for the area of the steel multiplied by the lever-arm of the resisting momen that is, the distance from the center of gravity of the steel to the center of com pression in the concrete. Similarly, in Formula (9), the denominator of th fraction is an expression for the area of the concrete in compression multipli by the lever-arm, x again being determined by Formula (4) and M being th maximum bending moment due to the external forces.

Reinforced-Concrete T Beams. Where beams or girders are used in rei forced-concrete building-construction there are usually accompanying floor-sla If these slabs are cast with the beams or girders they add very much to th strength of the latter, and when adequate bond and shearing-resistance are pro vided between the slab and the stem or beam, economical design requires th the slab shall be considered in determining the strength of the beam. The widt of slab that may be taken as part of the beam should not exceed one sixth th span-length of the beam, and the overhanging part on either side of the we or stem should not exceed six times the thickness of the slab. In any cas

ge must not be considered wider than the distance between the beams. ary floor-construction the spacing of beams, girders, and columns is gen- ı architectural or commercial consideration. Generally, the simplest pro- therefore, is to first determine the thickness of slab required for the given of beams, and this determines the thickness of the flange of the T beam. alculation of the girder, it is not objectionable to use the same slab, or of it as may be permissible, that has been used in the consideration of ı framing into that girder, as the compression-stresses, in the two cases, ght-angles to, and practically assist, one another. When, however, the slab-reinforcement is parallel to the girder, in the case of a combined m. and girder-construction, the slab-action produces compression in the ection as the girder-compression with a resulting overstress in the con- n this case, transverse reinforcement should be provided at right-angles rder and extending well into the slab.

ılas for Reinforced-Concrete T Beams. Fig. 17 shows a cross-sec- T beam resulting from the use of the slab as part of the beam, and

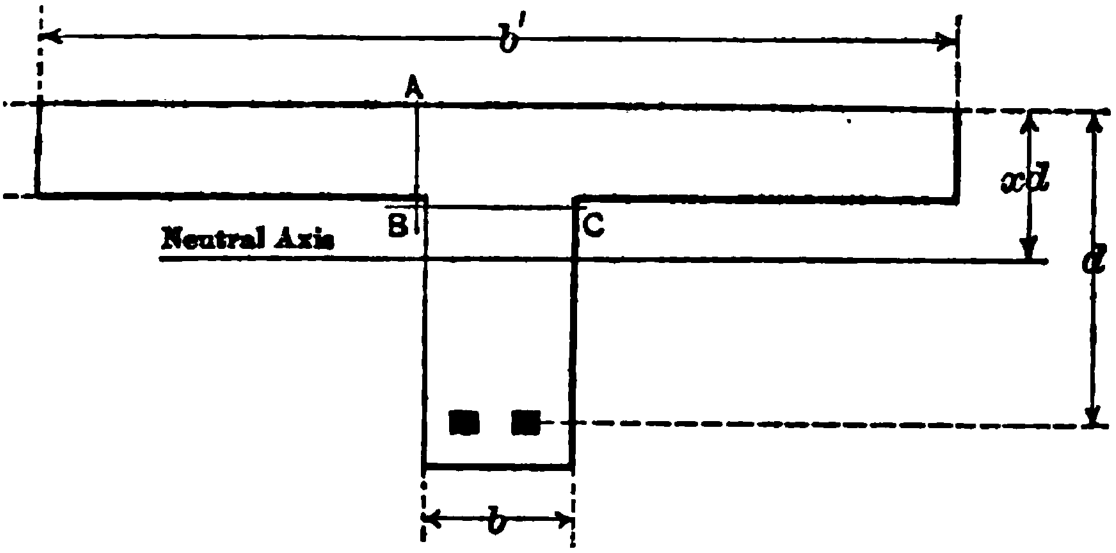


Fig. 17. Cross-section of Reinforced-concrete T Beam

ırly, also, the notation used in the formulas. In a construction of this : cases may be considered:

The neutral axis may fall below the flange, in which case

$$M = S_t A_s \left(d - \frac{t}{2}\right) \tag{10}$$

$$M = \frac{S_c}{2} b't \left(d - \frac{t}{2}\right) \tag{11}$$

formulas the small area of concrete in compression below the flange d and the center of compression is assumed to be at the center of This is done to simplify the formulas. The result is not materially ıd errs on the side of safety. The position of the neutral axis is given a (12)

$$x = \frac{2rd\,A_s + b't^2}{2d(rA_s + b't)} \tag{12}$$

›st economical percentage of steel by Formula (13)

$$p = \frac{S_c b't}{2\,S_t bd} \tag{13}$$

Case 2. The neutral axis may coincide with the under side of the flange, in which case

$$M = S_t A_s\left(d - \frac{t}{3}\right) \tag{14}$$

and

$$M = \frac{S_c b' t}{2}\left(d - \frac{t}{3}\right) \tag{15}$$

The economical value of p in this case is the same as in Case 1, Formula (13).

Case 3. The neutral axis may fall above the lower edge of the flange. This case is the same as Case 2, since for purposes of calculation all the concrete in the flange below the neutral axis is neglected and t becomes xd in this case as in the last.

Alternate Solution for Cases 2 and 3. In Cases 2 and 3 the section may also be considered as rectangular, with a depth d and a width b', and the formulas for rectangular beams, (1) to (9), may be used. Tables V, VI, VII, and VIII are also applicable in these two cases.

When the slab is considered an integral part of the beam, adequate bond and shearing resistance between the slab and the web of the beam must be provided. The concrete is ordinarily adequate to take the vertical shear through the flanges next to the stem, and is further strengthened by placing horizontal steel reinforcements across the top of the beam or girder, as described above. Whether or not the resistance to shear is adequate can be determined by the formula

$$S_s = \frac{S_h b\,(b' - b)}{2\,t} \tag{16}$$

in which S_s is the unit vertical shear at AB, and S_h is the unit horizontal shear at BC (Fig. 17). This should not exceed the safe unit shear for concrete unless steel reinforcement is provided. The value of S_h in the formula is

$$S_h = \frac{V}{b\,(d - \frac{1}{3}t)} \tag{17}$$

which, it will be noted, is the total vertical shear divided by the effective area of the stem.

Moduli of Elasticity. In the derivation of all these formulas and in the determination of the values of K, the ratio of the MODULUS OF ELASTICITY of the steel to that of the concrete plays an important part. It is necessary then to know what values to use. The generally accepted modulus of elasticity of steel is 30 000 000 lb per sq in. The modulus of elasticity of concrete varies with many conditions. Even in the same mixture, the character of the materials as well as the manner of mixing and placing, affect it. The modulus increases with the age of the concrete. It also increases with the richness of the mixture. It seems to decrease with an increase in the load on the concrete. It should also be noted that the modulus of elasticity as determined from a beam in flexure is greater than that determined from compression-cylinders. Moreover the modulus of elasticity as determined from compression varies with the point selected on the stress-strain curve. The different values for the RATIO OF THE MODULUS OF ELASTICITY of the steel to the modulus of elasticity of the concrete to be used in the design of reinforced-concrete construction, as fixed by the building regulations of various cities and by other authorities, is given in Table II, page 912. Values for the modulus of elasticity of concrete under different

for different mixtures determined by actual tests at the Watertow
e given in Table XI.

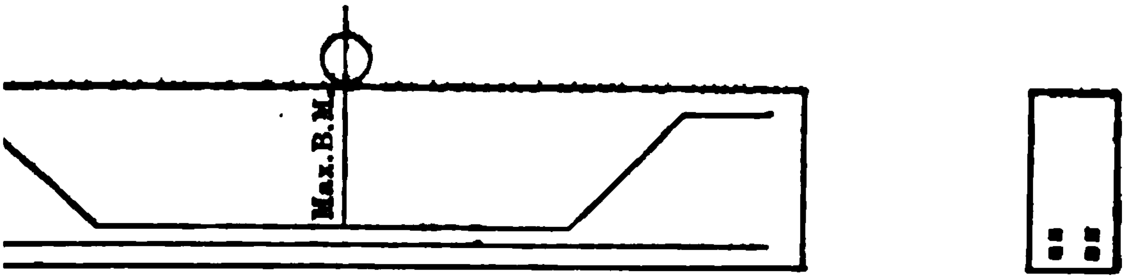
. Elastic Properties of Broken-Stone Concrete Twelve-Inch Cubes

position		Age	Modulus of elasticity in pounds per square inch between loads of			Tests made by
and	Broken stone		100 and 600 lb per sq in	600 and 1 000 lb per sq in	1 000 and 2 000 lb per sq in	
2	4	7 days	2 593 000	2 054 000	1 351 000	* Geo. A. Kimball.
2	4	1 mo	2 662 000	2 445 000	1 462 000	" " "
2	4	3 mos	3 671 000	3 170 000	2 158 000	" " "
2	4	6 mos	3 646 000	3 567 000	2 582 000	" " "
3	6	7 days	1 869 000	1 530 000	....	" " "
3	6	1 mo	2 438 000	2 135 000	1 219 000	" " "
3	6	3 mos	2 976 000	2 656 000	1 805 000	" " "
3	6	6 mos	3 608 000	3 503 000	1 868 000	" " "
6	12	1 mo	1 376 000	....	....	" " "
6	12	3 mos	1 642 000	1 364 000	....	" " "
6	12	6 mos	1 820 000	1 522 000	....	" " "

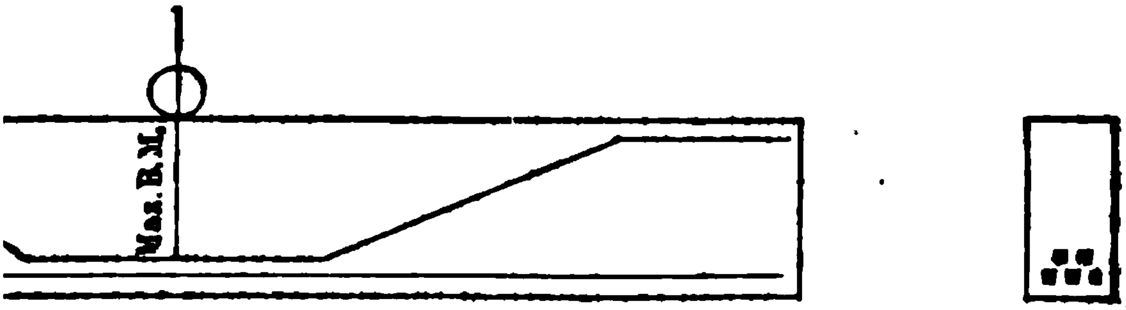
* Tests of metals, U. S. A., 1899, page 741.

g **Stresses.** The WORKING STRESSES for concrete and steel allowe
cities are given in Table II on page 912. In the determination
es of S_c, S_t, and r as taken from Table II are substituted in Formu
value of K may be taken directly from Tables V to VIII, pages 9
substituted in Formula (5). For M in that formula, the MAXIMU
OMENT due to the external forces is used.

; **Moments in Beams.** Beams and girders are usually consider
BEAMS, that is, as beams supported at both ends, but not built i



Reinforcement for Uniformly Distributed or Symmetrically Placed Load



9. Reinforcement for Unsymmetrically Placed Concentrated Load

r continuous, although in many instances they are actually carri
US BEAMS, over the supports. If continued over a support, th

is a NEGATIVE BENDING MOMENT at that support, and this negative bending m ment should be taken care of by reinforcements in the upper part of the bea This bending moment is one half that at the middle of a simple supported be loaded at the middle, and two thirds that at the middle of a simple support beam, uniformly loaded. In the case of simple supported beams loaded eith at the middle or with a uniformly distributed load, the bending moments crease toward the supports. For these reasons it is advisable in arranging t steel to be used for the tensional reinforcement, to select the bars or rods pairs, so that, as the supports are approached, a part of the reinforcement m be turned up toward the top and carried across the supports near the top as dicated in Figs. 18 and 19. For continuous beams and slabs with uniform distributed loads, the following are recommended for MAXIMUM POSITIVE a MAXIMUM NEGATIVE BENDING MOMENTS:

"For beams, the bending moment at middle and at support for inter spans, should be taken equal to $wl^2/12$, and for end spans it should be tak equal to $wl^2/10$ for middle and interior support, for both dead and live loads.

"In the case of beams and slabs continuous for two spans only, with their en restrained, the bending moment both at the middle support and over the midd of the span should be taken equal to $wl^2/10$."*

Beams simply supported at the ends must be considered as SIMPLE BEA with maximum positive bending moments equal to $wl^2/8$. In all the abo values, w is the load per linear unit and l the span in the same unit.

Bending Moments in Slabs. As floor-slabs are usually carried continuou across the supports, the maximum bending moment due to a uniformly distri uted load is assumed to be less than in beams simply supported at the en The New York City Regulations provide that "the bending moments at t center and at intermediate supports of floor-slabs continuous over two or mo supports shall be taken as $Wl/12$." The same regulations provide that "the ben ing moments of slabs that are reinforced in both directions and supported four sides and fully reinforced over the supports (the reinforcement passing in the adjoining slabs) may be taken as Wl/F for loads in each direction, in whi $F=8$ when the slab is not continuous or when continuous over one suppo and $F=12$ at both center and supports when the slab is continuous over bo supports." In these expressions W is the total distributed load and l the sp In square slabs with two-way reinforcement it is usually assumed that the loa ing is uniformly distributed and that half the load is carried by each system. rectangular slabs the amount of load carried by each system of reinforcement given by the formula

$$r = \frac{1}{2}\frac{W}{n} \qquad (1$$

in which r is the proportion of load carried by the transverse reinforcemen W the total load on the slab, and n the ratio of its longer to its shorter sid Using this proportion of load, each set of reinforcements is calculated as a sl with supports on two sides only, and the total number of rods required is d termined on the assumption of equal spacing. The rods may then be spac uniformly at the usual spacing for the central half of the slab and gradually r duced in number per foot of width to the edge of the slab, using one half many rods for the remaining two quarters. In this way, the amount of reinforc ment is reduced 25%. When the length of the slab exceeds the breadth by 50% the stresses in the longitudinal steel become so low that the construction

* Trans. Am. Soc. C. E., 1917, page 1127.

onomical. The slab should then be treated as one with a one-way reinforce-t.

rinkage-Stresses and Temperature-Stresses. In slabs resting on or ed over two supports some reinforcement should be provided at right-s to the tension-rods to provide against SHRINKAGE-STRESSES and TEM-TURE-STRESSES. Incidentally, this reinforcement may also serve to keep tension-rods properly spaced. In general it should not be less than one d of one per cent in amount and well distributed. It is common practice e from ¼ to ⅜-in rods, spaced about 2 ft apart. Deformed bars with irregu- surfaces and reinforcements of small diameters, placed as close as practi-le to the surface, are most effective.

The Disposition of the Steel. In designing the reinforcement for any m of loading, the full sectional area required must be provided at the point MAXIMUM BENDING MOMENT. As the supports are approached, part of the inforcement, as already indicated, is turned up, but care must be taken to keep so distributed that at any point there is still sufficient reinforcement below e neutral axis to furnish the necessary tensional resistance. The arrangement reinforcement for a uniformly distributed or symmetrically disposed load is own in Fig. 18, and for an unsymmetrically placed concentrated load, in Fig. 19. the first instance the maximum bending moment is at the middle of the am, the reinforcement is symmetrical about that point, and as much as one lf the amount of reinforcement may be turned up. In the second instance e maximum bending moment is at some other point than the middle, the rein-rcement must be so disposed that the full amount required will be under the ad or at the point of maximum bending moment, and the turning up must done between that point and the support. Other conditions might require s than half the reinforcement to be turned up. There is another reason for rning up the reinforcements toward the ends. In addition to the resistance the NEGATIVE BENDING MOMENT, there is a resistance to the SHEAR offered the metal running through the concrete at the points where the DIAGONAL NSION occurs.

The Percentage of Reinforcement. The AMOUNT OF THE REINFORCEMENT any case is determined by Formulas (3) and (13) for rectangular and T beams spectively. The values obtained by these formulas give the most econom-l amount. This may vary from ¼% to 1¼% of the cross-section area of ncrete, but will usually run about 7/10%. The nearest stock size of rods ving this amount or a slightly greater amount can be selected from the table ven on page 1514, or from the catalogues of the manufacturers of the various formed bars.* The NUMBER OF RODS used to make up the necessary ctional area must be determined by considerations mentioned in the follow-g paragraphs.

The Number of Reinforcing-Rods. As already suggested, an even number apts itself better to a symmetrical or balanced arrangement both in cross-sec-n and horizontal section. One rod does not permit of the turning up toward the pport. Two rods may be made either to continue along the lower edge of the am, or one may start at one support, run along the lower part and turn up be-nd the middle as it approaches the second support; and the second rod run milarly along the bottom from the second support and turn up after passing the iddle as it approaches the first support. Three rods may be arranged so that o continue along the bottom and the third, the middle one, turns up as it ap-oaches the supports. The arrangement for 4, 5, or 6 rods will naturally suggest

* See, also, paragraph on Commercial Sizes, page 915.

itself from what has been already said. Too large a number of rods is not sirable, as a large number of them together acts more or less as a screen for coarser particles of the concrete and prevents a close contact between it and steel. This matter of complicated reinforcement is one of considerable pract importance. If, however, the steel is satisfactorily incased with concret larger number of small rods is preferable to a small number of larger o The AREA OF CONTACT of a rod of smaller size is proportionately greater t that of a rod of larger size, as the perimeter varies directly as the diameter, the sectional area as the square of diameter of the cross-section. In order a rod may not slip, the ADHESION of the steel to the concrete must be equa or greater than the tension in the steel.

The Adhesion Required. The tension in a reinforcing-rod at any p having been determined from the given formulas, it must next be determi if, in either direction from that point, the AREA OF CONTACT of the stee large enough to make the total ADHESION equal to or greater than the TENSI If there is a deficiency in this respect it must be made up either by a mech cal bond or by anchoring the reinforcements at the ends. Safe VALUES ADHESION of concrete and steel are given in Table II, page 912. A safe to apply, without calculation, to the case of beams with a maximum bend moment at the middle, is to make the diameter of the rods not more than two-hundredth of the span. Under ordinary conditions, generally speaking, length of rod on either side of the point of maximum bending moment should at least eighty diameters for plain rods, and not less than fifty diameters deformed bars. Under unusual conditions the adhesion should be carefu studied. The apparent discrepancy between the first and second statement this paragraph is explained by an allowance made and based upon the fact the tension in the steel does not decrease uniformly with the decrease in tance from the supports. The allowance is purely arbitrary but is conside safe. For cases of unsymmetrically loaded beams it is best to examine ca fully into the conditions.

The Separation of the Rods. It has not been unusual in tests on bea to have the concrete split off from the under side along the line of the reinfo ment. This is due in part, if not entirely, to an insufficiency of concrete betw and around the reinforcement. To avoid this the SPACING or SEPARATION the reinforcing-rods in the cross-sections of the beams must be such that resistance of the concrete to SHEAR at the level of the rods is at least equal the ADHESION of the concrete to the steel. As a general rule the rods should spaced not less than two-and-one-half diameters on centers and about two ameters from the sides of beams. The clear distance between rods and the sp between rods and edges of beams should in no case be less than 1½ in. I formed bars, if stressed to their full tensional value, should be spaced fart apart than plain bars. At the middle of a beam, the BOND-STRESS is low, but the top of a continuous beam, over the supports, where the negative moment creases rapidly, the bond-stress is apt to be excessive and frequently limits diameter of the reinforcement.

Provisions against Shear or Diagonal Tension. Numerous tests of bea reinforced with horizontal rods without stirrups or inclined reinforcement h shown that DIAGONAL CRACKS occur when the maximum shear over the cro section is from 100 to 200 lb per sq in. Tests conducted on concrete with purpose of eliminating all other stresses but direct shear have given a SHEARI STRENGTH of concrete of from 800 to 1 600 lb per sq in. The ordinary conc beam has, therefore, a cross-section of sufficient area to withstand a SHEARIN STRESS of 200 lb per sq in. The cracks always occur at points where a la

G-STRESS exists in combination with MOMENT-STRESSES.
l loads, DIAGONAL-TENSION failure occurs under the co
a simple beam under a uniformly distributed load, the cr
supports. The inclination of the diagonal tension in the co
nt of two forces, changes, therefore, with the variations of

ams with horizontal rods only, that is, beams in which the w
ed by the concrete, the SAFE SHEARING VALUES to be used u
regulations are given in Table II, page 912. The SHEARIN
is determined by dividing the total VERTICAL SHEAR by th
ive depth, that is, the distance from the center of compre
the steel, by the width of the beam. The MAXIMUM SHEA
this case, not exceed 2% of the COMPRESSIVE STREN
When the resistance of the concrete to shear is not suff
ent must be provided by one of the following methods o
f them:

attaching to or looping around the horizontal members,
ertical members;
securely attaching inclined rods to the horizontals in such
prevent slipping;
bending of a part of the longitudinal reinforcement at ce
us providing against the diagonal tension and allowing
nount of horizontal steel to remain to resist the direct tens

tomary to use the calculated VERTICAL SHEARING-STRESS a
GONAL TENSILE or WEB-STRESSES. In all cases, the conc
carry its safe load, and it is ordinarily assumed that t
TAL VERTICAL SHEAR is resisted by the web-reinforcement.
with web-members, the total VERTICAL EXTERNAL SHEAR
ion should not exceed 6% of the COMPRESSIVE STRENGTH
e Building Code of New York City specifies that the SHEA
, when all the DIAGONAL TENSION is resisted by steel, shal
sq in. For beams in which part of the longitudinal reinf
n of bent-up rods, the MAXIMUM VERTICAL SHEARING-ST
3% of the COMPRESSIVE STRENGTH of the concrete.
sses in web-reinforcements may be determined by t

$$P = Vs/l$$

s inclined 45°, not bent-up bars,

$$P = 0.7\,Vs/l$$

is the horizontal spacing of the web-members, V the t
ar, l the effective depth from center of compression to ce
tress in a single reinforcing-member. Fixing the ALLOWA
6 000 lb per sq in, the spacing of web-members is expr
rmulas, when A is the cross-section of a web-member:

$$s = 16\,000\ Al/V$$

$$s = 16\,000\ Al/0.7V$$

ining the length of horizontals necessary to properly
sses, the same method may be employed as for plate

remainder of the bar being carried up as an inclined member and carried ov the top of the supports in continuous beams. The rods remaining at any poi at the bottom or top must be of sufficient sectional area to carry the direct tensi beyond this point. There must also be a sufficient length beyond this point prevent slipping. Web-members must be so spaced that there will be a reinforc ment intersecting every 45° line of rupture below the neutral axis. The Ne York City Code prescribes that the spacing of the web-members should n exceed three fourths of the depth of the beam in that portion where the we stresses exceed the allowable value for shear in concrete. Sufficient BON STRENGTH of web-reinforcement should always be provided in the COMPRESSIO SIDE of the beam. In SIMPLE BEAMS, that is, beams resting on two supports, t ends of the bars should preferably be bent into hooks. Where bent up throu large angles, web-members should extend horizontally along the upper part o beam for some distance.

Attached Shear-Members. Stirrups need not be firmly attached to the te sional reinforcement; but the allowable BOND-STRESSES and SHEARING-STRESS in the concrete must not be exceeded in transmitting the stresses between stirru and longitudinal rods. The stirrups and inclined members must also devel sufficient BOND-STRESSES to transmit the entire stresses for which they are d signed, and they must sometimes be supplemented with anchorages in the co pression-side of the beam. It is, perhaps, better to have them attached, they will certainly assist in anchoring the tensional reinforcement. Differe forms of stirrups and methods of attachment are used. In the Kahn syste (Fig. 11) the stirrups form a part of the tensional reinforcement. The U for either upright or inverted, is a very common form of stirrup, and may be a r of either round or square cross-section, or a flat strap as shown in Figs. 10 and 1 The Hennebique system employs both inclined rods and vertical stirrups. some cases, when the slabs and beams are constructed together, the slab-rei forcement is carried through the upper ends of the stirrups.

The Bond between Steel and Concrete. The BOND between the steel tension and the concrete must not exceed the safe working value. If the bo is not sufficient, the rod will slip. Tension-rods must, therefore, never be t large to develop sufficient BOND-STRENGTH to transmit the stresses. Wh bent-up bars are employed, the BOND-STRESSES in places, in both the straig and bent bars, will be higher than if all bars are straight. In cantilever bea the ends of the bars at the supports are fully stressed and the bars must carried into the supports and anchored to develop this stress. In anchori bars, an additional length must always be provided above that required, on t assumption of UNIFORM BOND-STRESSES. Wherever possible, adequate bo strength should be provided throughout the length of the bar in preference end-anchorage. Between plain bars and concrete the BOND-STRENGTH may assumed to be 4% of the COMPRESSIVE STRENGTH of the concrete.

The Breadth of a Reinforced-Concrete Beam of Rectangular Cros Section. The breadth of a rectangular beam, and of the stem of a T bea as already indicated, is generally dependent upon the amount of reinforceme necessary, and it is equal to the sum of the diameters of the tension-rods, t required spaces between them, and the amount of concrete outside of the ro needed to resist the shearing-stresses and to protect the steel. When no stirru are used in a beam it is necessary, also, to make the width of the concrete su cient to resist the horizontal shearing-stresses. This width should be at le equal to the sum of the perimeters of the tensional reinforcing-rods. T amount of concrete to be provided below the steel is fixed by the requireme for proper protection of the steel against fire and corrosion. (Pages 955–9

Compression-Rods in Beams and Girders. Steel reinforcement in the rm of rods is sometimes provided ABOVE THE NEUTRAL AXIS in beams and irders for the purpose of providing additional COMPRESSIVE STRENGTH where here is not sufficient concrete above the neutral axis to resist the total compression. If steel reinforcement is to be used for this purpose, the steel should e placed as high as possible, and the allowable unit compression in the steel mited to the actual compression in the concrete at that point multiplied by e ratio of the modulus of elasticity of the steel to that of the concrete, as in e case of columns with vertical reinforcement. The use of STEEL IN COMPRESSION in beams and girders, however, is not recommended, since at best it very uneconomical and the steel has a tendency to buckle and disrupt the oncrete.

Reinforced-Concrete Columns. Reinforced-concrete columns are of three neral types: (1) concrete with VERTICAL REINFORCEMENT near the outer rfaces; (2) concrete wrapped with SPIRALLY-WOUND WIRE or with metal ands; (3) concrete with a METAL CORE.

Lengths of Columns. The lengths of reinforced-concrete columns are variously limited by different authorities as follows, the figures being in each case e RATIO OF THE LENGTH to the LEAST LATERAL DIMENSION:

New York	15
Chicago	12
Philadelphia	15
St. Louis	15
Cleveland	15
Baltimore	16
San Francisco	15
Buffalo	15
Detroit	15

New York limits, also, the least side or diameter to 12 in, and San Francisco 10 in.

Vertically-Reinforced Columns. In determining the strength of columns th VERTICAL REINFORCEMENT, the steel is assumed to carry a load per square ch equal to the working load per square inch on the concrete times the ratio of e moduli of elasticity of the steel and concrete. The allowable stresses, ratio of oduli, etc., are given in Table II, page 912. For example, in New York a load 500 lb per sq in is allowed on the concrete, and 15 times 500, or 7500 lb per in on the steel, 15 being the ratio of the moduli, as fixed by the regulations, 1:2:4 concrete and steel. Not less than ½% nor more than 4% of vertical inforcement should be used in reinforced-concrete columns. The reinforcing-ds should be tied together horizontally at intervals of not more than the least e or diameter of the column. This prevents, to a great extent, the buckling the reinforcement under load and the consequent splitting of the concrete. e VERTICAL REINFORCEMENT, in order to serve its purpose of taking up the nding in the column, should be placed as near the outer surfaces of the column possible, consistent with proper protection of the steel. (See page 958.) If nsion is possible in the longitudinal steel, due to bending, the bars must be iced to resist the stress. In the DISPOSITION OF THE STEEL the same precautions are necessary as in the case of beams, in order to avoid a too close acing of the reinforcing-pieces or an excess of reinforcing-material. (See e 937.) As the concrete in columns is generally poured into the molds at e extreme top, it is particularly important to keep the interior free from erlacing steel across the column. In columns in which the steel is assumed

to furnish part of the COMPRESSIVE STRENGTH, it should be made continuo from the columns of one story into those of the stories below. The ro extending from one column may be connected with those above or below b means of pipe-sleeves.

Laterally-Reinforced Columns. Tests made on HOOPED CONCRETE COLUM at the University of Illinois in 1907, at the Watertown Arsenal in 1906, and the University of Wisconsin in 1906 and 1907, show that the ultimate compressi strength of such columns is increased from 500 to 1 000 lb per sq in for ea percentage of hooping employed. The increase of strength is due to the LATER COMPRESSIVE STRESSES developed by the restraining action of the hoops bands at right-angles to the direct compressive stresses. Below the limit elasticity, however, very little stress is developed in the lateral steel, and t tests show that at an early stage, the deformation or shortening of the colu is equal to that of plain concrete. With further loading, the laterals begin work and prevent failure, thus increasing the so-called TOUGHNESS of the colu and the ultimate compressive or breaking strength. This effect has been v riously allowed for by considering the hooping-metal equivalent to and repla by imaginary longitudinals. Considère and other investigators have shown t the hooping is equivalent to 2.4 times as much longitudinal steel. It is gen ally conceded that hooping permits of a somewhat higher unit stress in the c crete. The New York City Building Code permits an axial compression in s columns, having not less than ½% nor more than 2% of hoops or spirals spa not farther apart than one sixth the diameter of the enclosed column nor m than 3 in, and having not less than 1% nor more than 4% of vertical reinfo ment, not to exceed 500 lb per sq in on the concrete within the hoops or spi nor 7 500 lb per sq in on the vertical reinforcement, plus a load per square i on the effective area of the concrete equal to twice the percentage of late reinforcement multiplied by the permissible tensile stress in the lateral reinfo ment. St. Louis and Cleveland permit 2.4 times the volume of hooping to considered as longitudinal reinforcement; Chicago 2.5 times; and Cincinnati times.

New York Requirements Expressed by Formulas. The safe lo for reinforced-concrete columns according to the requirements of the N York Building Code may be determined by the following formulas, in wh

W = total safe load, in pounds;

A_c = the effective cross-sectional area of concrete, in square inches, which the case of columns with longitudinal reinforcement only, may taken as the entire area, and in the case of hooped columns is limi to the area within the hoops;

A_s = the cross-sectional area of the longitudinal steel, in square inches;

p = percentage of lateral reinforcement (hooping), that is, the volume of hooping divided by the volume of the concrete enclosed within hooping, for each unit length of column;

S_c = allowable compressive stress in the concrete, in pounds per square i which is taken at 500 for 1 : 2 : 4 concrete, and at 600 for 1 : 1½ concrete;

S_s = allowable compressive stress in the steel, in pounds per square i which is taken at 7500 when 1 : 2 : 4 concrete is used, and at when 1 : 1½ : 3 concrete is used;

S_h = allowable tensile stress in the hooping steel, in pounds per square i which may be taken at 35% of the elastic limit, but not more 20 000.

longitudinal reinforcement only,

$$W = A_c S_c + A_s S_s$$

steel must not be less than 1½%, nor more than 4% of the and the reinforcement must be secured against displacement paced not farther apart than 15 diameters of the vertical 12 in.

is the safe carrying capacity of a 12-in square column of inforced in each corner with ⅞-in square bars?

rea of the concrete may be taken at $12 \times 12 = 144$ sq in. ias a sectional area of 0.7656 sq in. The area of the steel i sq in, a little over 2% reinforcement. The allowable and steel are 500 and 7500 lb per sq in, respectively. Hence

$$14 \times 500 + 3.06 \times 7\,500 = 94\,950 \text{ lb} = 47\tfrac{1}{2} \text{ tons}$$

ns,

$$W = A_c S_c + A_s S_s + 2\,p\,A_c S_h$$

inforcement not less than 1%, nor more than 4%, and n ½%, nor more than 2%, the hooping being spaced not ne sixth of the diameter of the enclosed column, and at

ine the maximum load that should be placed on a 24-in 1½ : 3 concrete, with spiral hooping of 5⁄16-in cold-drawn pitch, and reinforced longitudinally with six 1-in round just inside the hooping, and fastened to it, the concrete ide the hooping.

ective sectional area of the concrete has a diameter of re $20 \times 20 \times 0.7854 = 314.16$ sq in. For an inch in height, the concrete is 314.16 cu in. The area of a 1-in round The area of longitudinal steel is $6 \times 0.7854 = 4.71$ sq in, ross-sectional area of 5⁄16-in wire is 0.0767 sq in. The wire is 62.75 in, and as the turn is made in a height of its for an inch of height of the column is $\frac{1}{3} \times 62.75 \times 0.0767$ ½%. The working stress per square inch for the con- longitudinal steel 7 200 lb, and for the hooping 20 000 lb.

$$600) + (4.71 \times 7\,200) + (2 \times 0.005 \times 314.16 \times 20\,000)$$
$$= 285\,240 \text{ lb, or } 142.6 \text{ tons}$$

Reinforcement. At the top or base of the columns in ping should be made to continue through the floor-con- tain conditions, when the floor-construction is practically mns, thus affording good lateral support, equal to the better to omit the wrapping and avoid the possible com- orcement from column, girder, and floor-construction and ng of the bond of the concrete. The materials used for the teel wire or steel bands. When wire is used it is spirally s through the full length of the column. The ends of the he column and turned down to such an extent that when poured and set, there will be sufficient anchorage to resist

the tension in the wrapping due to the outward pressure of the concrete. W metal bands are used, as in the Cummings system, care must be taken to m the riveted joints in the bands as strong as the bands themselves. A for wrapping that has the merits of rapidity and ease of erection is shown in columns used in the Bush Terminal Warehouse, Borough of Brooklyn, York City, described on page 958.

Fig. 20. Concrete Column with Steel Core

Metal-Core Columns. The object of this type of column is to pro construction for tall or heavily loaded buildings that will have the ne strength and yet not encroach too seriously on the floor-space. For th of column some engineers advocate placing a steel core through the axi concrete, the steel taking the bulk of, if not all of, the load.* "A ration of design is to determine the strength of the steel column by the use of the c formula for the proper l/r of the column and to consider the concrete of th

* Trans. Am. Soc. C. E., Vol. XIV, Part E, page 556.

› have a stress-value proportional to the strength of plain con-

n H. Burr designed a column (Fig. 20) for the McGraw Building, New . The steel core has sufficient strength as a column, independent of ete, to carry the entire dead load coming upon it, the stresses in the ; in no case greater than those allowed on steel columns under the Building Code, considering the ratio of length to radius of gyration. re, those stresses were not allowed to exceed 9 000 lb per sq in in any : live loads were provided for by placing enough concrete within the work to prevent the stress on the concrete from exceeding 750 lb This is one twelfth of the maximum allowable load on the steel. te outside of the steel was considered only as a protection against fire ion. Columns of this type should be designed with caution. The ould not be relied upon to tie the steel units together or to transmit m one unit to another. The units should be tied together by tie- attice-bars in conformity with the standard practice for structural For high percentages of steel, the concrete will develop low unit d caution should, therefore, be used in placing dependence upon the

ixtures of Concrete. Increasing the proportion of cement in a reases the ultimate strength of the concrete proportionally and is designing columns with smaller cross-sectional area. The increased : strength is also accompanied by a higher modulus of elasticity. e, the employment of a rich mixture also permits of higher propor- ses in the steel and consequently a more economical design. The sses in a monolithic member, however, may be considerably com- the excessive shrinkage of rich mixtures, which have a tendency to : New York Building Code provides that "in concrete columns the on the concrete may be increased twenty per cent when the fine and gates are carefully selected and the proportion of cement to total increased to one part of cement to not more than four and one-half egate, fine and coarse, either in the proportion of one part of cement, -half parts of fine aggregate, and three parts of coarse aggregate, oportion as will secure the maximum density. In such cases, how- npressive stress in the vertical steel shall not exceed seven thousand l pounds per square inch."

gs Lateral Reinforcement. Robert A. Cummings of Pittsburgh, Welding Company), following a European practice, has applied a einforcing compression-members by placing horizontal wire spirals ight-angles to the main compressive stresses. This practice is based y that the failure of a concrete prism will take place along lines e direction of the applied load. The method has been very success- ing the heads of precast concrete piles, driven by hammer.

Columns. When a building for any reason need not be treated of structure, space and time may be saved by using CAST-IRON or NS. In such cases the column-connections must be designed with ings for the concrete construction and so that there will be a CON- hat construction; for the great advantage in reinforced-concrete lies in its MONOLITHIC character. When cast-iron columns are ds of the columns may be cast with openings through which the

* University of Illinois Bulletin, No. 56, 1912.
† Proc. Am. Soc. C. E., Feb., 1913, page 153.

reinforcement may pass from one side to the other. Fig. 21 shows how this been done in a building at Gay and Christopher Streets, New York City w out impairing the strength of the columns at the connections.

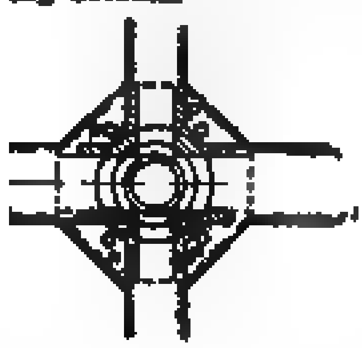


Fig. 21. Connections for Cast-iron Columns and Reinforced-concrete Constructi

Steel Columns. In STEEL COLUMNS it is simpler to provide connec between the reinforcing-rods and the steel shapes of the columns. When reinforcement does not go through the columns, some rods should be pl outside of them to tie as much as possible the concrete on one side to tha the other.

Eccentric Loads. Bending stresses due to LATERAL and ECCENTRIC L must be computed so that the combined direct and bending stress does exceed the allowable maximum stress for axial compression. Formulas eccentric loading on columns are given in Chapter XIV, pages 453 and 48

Concrete Walls. If not reinforced, concrete walls are generally requir be of the same thickness, for given conditions, as brick walls. Under such cumstances they are not as economical as brick walls. If reinforced and us bearing-walls, they can be reduced to about two thirds the thickness of walls, provided, however, that the load on the concrete does not exceed the load per square inch permitted on reinforced columns. The ratio of unsupp height to thickness should not exceed that fixed for columns. For span walls, supported entirely on girders, the minimum thickness should be Such walls should be reinforced with not less than ½ lb of steel per square of wall, in the form of rods placed vertically and, less frequently, horizon

Reinforced-Concrete Footings. (See, also, pages 186, 225, and 226.) principles underlying the design and construction of reinforced-concrete foo are the same as those applied to other types of footings. In wall, pier, or col footings, the overhang or off-set must be considered as an INVERTED CANTIL loaded uniformly with a load per square foot equal to the load per square imposed on the underlying soil. The reinforcing-rods will then necessaril placed near the lower surface of the footing and the size and number determ as shown on pages 937, 938, and 979. A detail often overlooked in reinforced

ngs is the tendency to SHEAR at the edge of the wall, pier, or column . When footings would otherwise become very ECCENTRIC, cantilevers resorted to, the same as for steel construction. (See pages 165 to 169, o 982.) The maximum bending moments are determined and the signed as described on pages 925 to 932, and 978 to 982. Steel in hould be protected by at least 3 in of concrete.

ry of Reinforced-Concrete Footings. Great economy over steel-other types of footings may often be effected by the use of REIN-NCRETE FOOTINGS. The cost of the latter type will vary from 20 to e cost of a corresponding STEEL-GRILLAGE FOOTING. This difference counted for. The amount of excavation for the reinforced footing is much less than for the steel grillage. A smaller amount of concrete d this concrete is considered in the calculations for strength; whereas l grillage, the concrete is chiefly provided for incasing and protecting The amount of steel is much less, being used only to supply the ten-tance of the construction, the compressive strength being supplied ch cheaper material, concrete. Incidentally, the protection of the reinforced footing is generally more certain than in the steel-grillage

e Piles. Concrete piles are discussed in Chapter II, pages 196 to me of the types are there described.

ions in Reinforced-Concrete Construction. Much good judgment played and must be exercised in the design of the details in these . The great value of reinforced-concrete construction over other e possibility of securing great RIGIDITY. This can only be attained esult is as nearly MONOLITHIC as possible. We than have mass to ration and this advantage, in the case of workshops or factories in e is machinery, is readily seen. The reinforced-concrete buildings hrough the severe San Francisco earthquake in May, 1906, in good were those in which attention had been given to the details and con-To secure a monolithic character requires CONTINUITY not only in the it also in the reinforcement. This often means that there is a net-l at the connections. If this is carried to excess, the BOND and CON-the concrete is apt to be broken, even when the spaces between the are thoroughly filled. But when there is such a network of steel ike a sieve and the spaces are not readily filled. For this reason it e a richer mixture at the columns and to keep the aggregate as small The connections of floor system to columns are particularly trouble-respect, and partly for this reason and partly to insure rigidity, uld be provided under the girders at the columns, with metal rein-ear the inclined surfaces of these brackets.

ed-Concrete Stairs.* Some of the most interesting work that has e reinforced concrete has been the construction of stairs. The rein-eing in the form of comparatively small, limber bars, can be adapted y shape for which molds can be constructed, and when a wet, rich h small aggregate is used, little or no difficulty need be experienced As an example of such work, the stairs in the residence of G. W. in New York City, may be cited. When these stairs were five test of their strength was made, without distress, by dropping a ur bags of cement, weighing about 380 lb, from the floor above to

* See, also, pages 900 and 983.

the intermediate platform, a distance of 11 ft. No injurious effects were ticed.*

4. Types of Reinforced-Concrete Construction †

Mill-Construction. In localities where the cost of labor is high and wl the conditions cause more or less congestion, it is probably more economica use brick instead of concrete for the walls. In such cases the type of const tion is similar to ordinary MILL-CONSTRUCTION. Provision must be made to chor the beams and girders, and this can be done by bending the ends of reinforcing-rods so that they will extend horizontally into the walls on each s

Skeleton Construction. The SKELETON TYPE of construction seems to the form best adapted to reinforced concrete. A framework of columns, gird beams, and flooring is built, as in steel construction, the wall-girders and colum of course, being designed to carry the weights of the outside walls as wel that part of the floor-loads and live loads which comes on them. The work this type of construction, can generally progress more steadily than in the m CONSTRUCTION since the concrete work need not be stopped at any time to for the brickwork to be carried up, if brick is used for the walls. In the SKE TON CONSTRUCTION any type of outside wall may be used; brick, concrete, etc. In some cases the panels are simply filled in with brickwork, 8 or 1 thick, leaving the concrete columns and girders showing between the b panels. For walls situated on property-lines where adjoining buildings are li to be erected, this is not objectionable. If the wall remains exposed and a g appearance is a consideration, the columns and girders can be treated archi turally to set off the brickwork; or the brickwork may be continued as a fa over the outside of the columns and girders. This was done in some of the B Terminal Warehouses, Borough of Brooklyn, New York City.‡ To thoroug secure this brick facing, galvanized anchors were placed in the concrete colu and girders as they were erected, projecting sufficiently to bond into the br joints. In using concrete for the panels the sides of the columns are cast w pockets, grooves, or recesses to receive the panels, which, as in the case of bri work, are most satisfactorily and most economically built after the remova the molds from the skeleton frame. In the Marlborough-Blenheim Hotel Atlantic City, N. J., the panels are filled in with hard-burned terra-cotta t and a stucco applied on the outside. This makes a comparatively light c struction and affords good insulation. The particular advantage in the SKE TON TYPE of construction, especially for workshops and factories, is the possibi of large window-areas affording light and ventilation.

System M. A type of construction known as System M has been develo by the Standard Concrete Steel Company of New York City (Fig. 22). It c sists of a light steel skeleton frame designed to carry the dead load of the en structure, except that the columns are designed to carry the gross loads. T structure is incased in concrete, making ultimately a reinforced-concrete constr tion.§ Its advantage consists in its adaptation to the erection and inspectio the steel reinforcement before even the centers or molds are placed in positi Under congested conditions, such as prevail in large cities, it is a rapid form

* For a detailed description, see Cement, Jan., 1904, and Engineering Record, Dec, 1903. For other examples of stair-construction, see Engineering News, June 30, 19

† See, also, Chapter XXV.

‡ For a description of this building, see Engineering Record, March 3, 1906.

§ For fuller description, see Engineering News, April 25, 1907, and Engineering Rec June 22, 1907.

on. The use of the steel in this type is, however, not economical
o get the necessary strength in the steel framework, shapes must be
h do not offer the amount of adhesion that should result from the

Fig. 22. System M Type of Reinforced-concrete Construction

metal used. Furthermore such shapes must necessarily be subjected
ending, which tends to break the bond between concrete and steel.
ab Construction.* In this form of construction beams and girders
ated almost completely, if not entirely, and the slab is made to rest
the columns; the tops of the columns are enlarged into extended caps.
m of construction employs a shallower floor construction than is ordi-
inable. The floor-centering, too, for purposes of erection, is somewhat
pecially in those forms of slabs in which the lower surface is all in one
e slab may be of uniform thickness between the edges of the column-
a portion of it, symmetrical about the columns, may be thickened to
UMN-DROP, or the slab may be thickened to form a band or shallow
een columns, with a paneled ceiling at the center of the panel.
the method of reinforcing the slab and columns, a number of systems
developed which may be divided into four general classes: (1) the
SYSTEM; (2) the FOUR-WAY SYSTEM; (3) the THREE-WAY SYSTEM; and
CUMFERENTIAL SYSTEM. In the TWO-WAY SYSTEM, the reinforcement
direct bands between the columns in both directions, with an interior
wo-way rectangular bands on the remaining panel-area at the center.
R-WAY SYSTEM, the reinforcement is placed in two direct bands in the
gular directions, and in two diagonal bands which cross the panel
between columns. In the THREE-WAY SYSTEM the reinforcement
bands directly connecting the columns and passing over the column-
the CIRCUMFERENTIAL SYSTEM, circumferential reinforcement is
nd the columns, with bars radiating from the column-centers. Con-
s of reinforcement are also placed at the center of the sides joining
ters, which overlap the circumferential reinforcement at the columns,
nter of the panel is reinforced in a similar manner. Some of the
veloped are modifications of the above or a combination of two or more
ral types described. The principles of design are based on empirical
termined by extensometer tests made on completed buildings.
ystem. This is a TWO-WAY SYSTEM developed by the Condron
Chicago, Ill. It is constructed either with a slab of uniform thick-
a drop-panel, or with a paneled ceiling. Each panel of the slab is

* See, also, Girderless Floors, Chapter XXV, page 993.

divided into two sets of strips, called the MAIN-SLAB STRIPS and the MID-
STRIPS, which are designed as flat, shallow beams.

Corr-Plate System. This type of construction has been developed an stalled by the Corrugated Bar Company, Buffalo, N. Y. The constructi similar to other TWO-WAY SYSTEMS and is installed either with or without d The reinforcement is distributed across the entire slab, with varying spaci resist the stresses determined experimentally.

Mushroom System. The MUSHROOM SYSTEM, invented by C. A. P. Tu Minneapolis, Minn., is one of the earliest of the flat-slab constructions. striking and essential feature which gives this system its name is the gra spreading out of the column at the top to form a cap, the diameter of whi seven sixteenths of the sum of the distances between columns in the directi the sides of the slab. The longitudinal column-reinforcement is bent to fo the curved outer surface of the cap, and the cap is reinforced both radially circumferentially. The slab-reinforcement is placed at the top of the slab the columns and allowed to sag to a catenary curve with the low point nea bottom of the slab at the middle of the span. The thickness of the slab v from 1/35 to 1/40 of the shorter distance between the column-centering. Th essentially a FOUR-WAY SYSTEM with the added features described above.

The Cantilever Flat Slab, designed by the Concrete Products Comp Chicago, Ill., is another type of FOUR-WAY SYSTEM. It differs from the on scribed in the preceding paragraph mainly in the construction of the column The column-bars are not bent to the shape of the cap but continue up stra The horizontal cap-reinforcement is provided by a shop-made frame of r bars, held together by a Diamond Bar which is intended to resist the cir ferential stresses. The diameter of the cap is about 3/10 the span and the t ness of the slab about 1/35 the span. Whenever necessary to provide for l shearing-stresses and bending-stresses around the column, the slab is incre in thickness at that point, forming, in appearance, an extended cap at column-head. Later extensometer tests have proved the use of radial with rings around the column-heads to be inefficient, and they therefore been abandoned.

Three-Way System. This system was invented and patented by Davi Morrow, Cleveland, Ohio. The columns are located at the apexes of equila triangles, making equal the bands of steel between the columns. The reinf ment over the columns is placed in three instead of the four layers of the f WAY SYSTEMS. Flaring circular caps, with hexagonal or circular drops, are vided over the columns.

S. M. I. System. This system was invented and patented by Edw Smulski, and is controlled by the S. M. I. Engineering Company, Boston, M Circumferential and radial reinforcement is placed in both the top and bot of the slab, with trussed bars extending both rectangularly and diagonally tween the columns (Fig. 23). The radial bars are provided with a semicirc hook to transfer the stresses into the concrete by bond, and to engage the ri reinforcement in the center of the panel. To prevent cracking on the top o slab between columns, additional short, straight bars are sometimes used.

Patents for Flat-Slab Construction. In 1901 and 1902 patents granted to O. W. Norcross, covering girderless floor-construction reinforced bands of wire netting extending from column to column. Application fo original C. A. P. Turner patents was made in 1905. In 1915 the United St Courts held that the Norcross patents covering girderless floors were fu mental, and that bands of bars were, to all intents and purposes, the sam

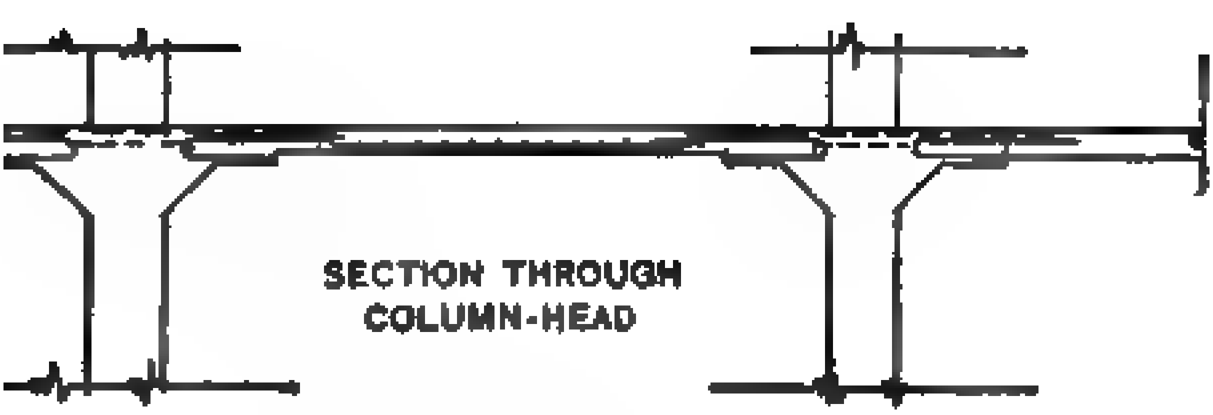


Fig. 23. S. M. I. Flat-slab System

e netting. It would seem, therefore, that any system of floor-con- pending upon bands of bars running either diagonally or crosswise to column constitutes an infringement of the Norcross patents. promoters of flat-slab construction in the United States are now er the Norcross patents; but several other United States patents anted covering special methods of construction and reinforcement.

tion Hollow-Tile and Reinforced-Concrete Construction. In nimize the cost of centering, the floor-construction shown in Fig. 24 vised. It consists of a series of reinforced-concrete beams with between them of the width of the hollow-tile blocks. In erection, a

flat centering is used, which, however, need not even be continuous. Planks few inches wider than the concrete beams, are placed under the spaces to be fil by the beams, and the tiles are laid in rows and supported along their edges the planks, thus forming the sides of the molds for the beams. The reinfor ment is placed and the concrete poured, with or without floor-plates, as the nec sities of the case may require. Care must be taken in pouring the concr

Fig. 24. The Combination Tile and Reinforced-concrete System

that the tiles are not displaced sidewise. The tiles should fit closely at th joints, otherwise the finer particles of the concrete are liable to flow into the either making the concrete porous or requiring more cement and sand than necessary. This form of construction, besides being economical in centeri offers the advantages of a flat ceiling without the application of lath and, in ro construction particularly, of freedom from condensation.

The Floretyle Systems. A floor-construction similar to the hollow-t construction just described has been devised by the Truscon Steel Compan Youngstown, Ohio, in which forms of corrugated sheet steel, called FLORETYLE replace the hollow tiles. The Floretyles are furnished in lengths of 28½ and in, and in depths of 6, 8, 10, and 12 in. The width at the base is 20¼ in, with t sides tapering at an angle of 7° 30′. They are furnished in two types, either w serrated edges for use with the company's Hy-rib lath for ceilings, or with strai edges for use where paneled ceilings are required. Other makes of metal for used in the construction of reinforced-concrete floors are the G F STEEL TILES of t General Fireproofing Company, Youngstown, Ohio, and the WISCOFORMS of t Witherow Steel Company, Pittsburgh, Pa. Besides a reduction in weight finished floors, the additional advantages in the use of these steel tiles or for over the terra-cotta fillers are: Greater economy in centering, larger coveri capacity, less bulk in shipment because the forms can be nested, and less dam

	6 in	8 in	10 in	12 in	14 in.
Average weight, in pounds per square foot . . .	40.100	46.000	53.500	61.000	72.60
Cubic feet of concrete per square foot of floor	0.278	0.319	0.371	0.422	0.50
Core-area, percentage of section	58.300	61.700	63.000	63.800	62.20

on of water from the concrete and of the flooring-out of the cement at A concrete floor of O F STEEL TILES, spaced so as to make 4-in con-s, 24 in on centers, and having 2 in of concrete above the tiles, with- is said to have the properties shown in the table on page 952,) the depth of tiles used.

y Tile Systems. Similar in general principle but using reinforce- directions, at right-angles with one another, is the COMBINED HOL-ID-CONCRETE FLOOR, controlled by the Burchartz Fireproofing Com- New York City, under the Burchartz patents (Fig. 25). This system ra-cotta blocks, channels, and soffits, providing uniform flat ceilings ster can be applied without the use of metal lath. In this case the ilated as a slab supported on four sides. (See page 936.) The con- :vented from running into the hollow spaces of the tile by the use -cotta channels as shown.

ies. In the FLOREDOME-CONSTRUCTION, put on the market by the el Company, Youngstown, Ohio, the tile spacing-blocks of the TWO- STEM are replaced by rectangular dome-shaped steel forms with the pen. Lightness in floor-weight, ease and rapidity of installation, and s are the advantages claimed. The ceiling-treatment in this con- similar to that in the Floretyle system. The base of the domes is .5 in square; the depth varies, being 6, 8, 10, or 12 in.

of Combination Systems. While the tiles may under favorable ld to the strength of the COMBINED FLOOR-CONSTRUCTION, the chances tory workmanship are too great to consider them in the calculations In the floors reinforced in one direction, the construction should s a series of either rectangular beams, or T beams, as the concrete :r to or above the top face of the tiles. The TWO-WAY REINFORCED N should be treated as if it consisted of a slab supported on four . series of intersecting rectangular beams, or T beams. If the con- o be treated as a series of T beams or as a slab, the concrete should t 2½ in above the top surface of the slab and the tiles or fillers should % of the volume of the construction.

y-molded Construction. The UNIT OR SEPARATELY-MOLDED CON-nsists of precast reinforced-concrete members, columns, girders, bs, either molded at the site of the building, or made at the factory to the site ready for use. The various members are swung into h the same manner as steel is erected, and fitted together in the interlocking reinforcement and poured grouting. Great economy this method of erection on account of the saving of forms. Maxi- y, however, cannot be obtained on a building operation of less than as economy is obtained by the greater use of the forms and the : the erecting-crews with the particular type of building. But onditions, economy can also be shown on an operation involving 000 sq ft. The advantages of this construction are said to be: nber of uses possible of one set of forms, especially on a large opera- ll number of men required, due to the extensive use of locomotive- trucks, derricks, etc.; the ease with which the units may be in- being poured and before entering the building; and the fact that all kes place before the units enter the structure, thus eliminating cks in the building. The disadvantage of such a system, however, e in the lack of sufficient rigidity in tall, separately-molded unit All floor-members must be designed and cast as simple, NON-CON-TS, with the reinforcement left projecting at both ends to serve for

Fig. 25. Republic Two-way Tile-and-concrete System

tructure together. These junctures are made after the units are place, and supported by a pouring of rich concrete. For tall strucore feasible to erect a light structural-steel frame and employ the -units only. (See Chapter XXIII, page 854.) The saving in cost ticularly in low buildings, and more especially in one-story strucbuildings having been erected at a saving of from 10 to 20% over construction. Methods of interlocking the units and providing details are constantly being improved and a series of tests of the such connections is being carried on by the Unit Construction St. Louis, Mo. There are under construction, or already completed, igs of this type installed by the above company, including five-story · the National Lead Company, at St. Louis, Mo., Kansas City, ttsburgh, Pa.; a three-story building for the Ohio Cultivator Comeville, Ohio; five acres of car-barns at Philadelphia, Pa.; and approxteen-and-a-half acres of cotton-warehouses at Memphis, Tenn. ie Engineering Company of New York City has erected five-story buildings with its Unit system, in Boston, Mass.

'ire-Resistance of Reinforced-Concrete Construction

luctivity of Reinforced Concrete. Concrete is a poor conductor in this fact lies whatever virtue it has as a fire-proof material.† A made by Professor Woolson of Columbia University, New York orted at the 1907 meeting of the American Society for Testing Matethe following results:

all concrete mixtures when heated throughout to a temperature 500° F. will lose a large proportion of their strength and elasticity, fact must be well remembered in designing."

all concretes have a very low thermal conductivity, and therein known heat resisting properties."

as a result of this low thermal conductivity, two to two and one- concrete covering will protect reinforcing metal from injurious eriod of any ordinary conflagration (provided, of course, that the in place during the fire)."

reinforcing metal exposed to the fire will not convey by conducious amount of heat to the embedded portion."

the gravel concrete was not a reliable or safe fire-resisting aggre-

rength of Reinforced Concrete. If its NON-CONDUCTIVITY were olved in the fire-proof character of concrete, the minimum thickfor the protection of the steel could be easily determined. But of the concrete is more or less affected when exposed to extreme rt has been made to determine this effect and a summary § of the orted by Professor Woolson of Columbia University, New York in Tables XII and XIII.

Record, Vol. 60, page 643; Engineering News, Vol. 58, page 5; Proal Association of Cement Users, 1910, page 391.

remembered that in this and succeeding paragraphs on the fire-resisting ncrete, only such material as is used in reinforced concrete, is considered. nder concrete as a fire-proof material is discussed in Chapter XXIII,

News, Aug. 15, 1907, page 168.

oc. for Test. Mats., Vol. IV, page 433.

Fire Tests on Reinforced Concrete. The EFFECT OF FIRE on reinfor concrete has been studied in a number of tests made by the building authorit of New York City and Philadelphia, and in some of the conflagrations in t country, notably at San Francisco. The tests to which the sample full-s constructions have been subjected are similar to the test described in Chap XXIII, page 827.*

Table XII. Tests of Concrete Blocks Heated on All Sides †

Specimens, 6 by 6 by 14-in prisms; proportions 1 : 2 : 4
Age 2 months; temperature 1500° F.

Treatment	Aggregate			
	Limestone	Trap-rock	Cinder	Gravel
Modulus of elasticity,				
At 200 lb per sq in:				
Unheated	6 000 000	3 430 000	1 090 000	8 000 00
Heated 3 hours	200 000	150 000	49 500	....
Heated 5 hours	....	129 000	571 000	....
At 400 lb per sq in:				
Unheated	6 000 000	4 355 000	960 000	6 887 00
Heated 2 hours	285 000	222 000	....	....
Heated 5 hours	....	188 000	....	....
At 800 lb per sq in:				
Unheated	5 647 000	4 355 000	915 000	6 000 00
Heated 3 hours	425 000	348 000	....	....
Breaking-load in lb per sq in:				
Unheated	2 740	3 140	1 400	2 7[illegible]
Heated 3 hours	1 345	1 400	547	....
Heated 5 hours	870	997	504	....

Table XIII. Concrete Blocks Heated on One Face Only ‡

Specimens, 6 by 6 by 14-in prisms; proportions 1 : 2 : 4
Age 2 Months; temperature 1 500° F.

Treatment	Aggregate	
	Limestone	Trap-rock
Modulus of elasticity. (Blocks heated 5 hrs.)		
At 200 lb per sq in	293 400	200 000
At 400 lb per sq in	521 700	268 000
At 800 lb per sq in	730 700	379 000
Breaking-load in lb per sq in	1 840	1 705

* For a partial list of these tests, see Table in Proc. Am. Soc. for Test. Mats., Vol. V page 128. Several tests have been made since that report was submitted.
† Proc. Am. Soc. for Test. Mats., Vol. VI, page 446.
‡ Proc. Am. Soc. for Test. Mats., Vol. VI, page 448.

lusion, from a study of the tests in detail,* shows that to a depth
bout 1 in, the concrete is seriously impaired and easily washed off by a
applied to the surface. Any stone containing an appreciable per-
arbonate of lime will calcine and may cause failure. Where the con-
poorly designed, allowing an excessive deflection, the fine cracks
ete below the steel will open to such an extent as to permit the heat
metal reinforcements. When the reinforcement is such as to pro-
of weakness in the concrete there is liable to be a flaking off of the
l a consequent exposure of the metal.

ire Tests of Reinforced Concrete. The EARLIEST TEST of a re-
crete building in an actual fire occurred in 1902, in the four-story
e Pacific Coast Borax Company, at Bayonne, N. J. The roof of
was of wood, and with the contents of the building, was destroyed
The only damage suffered was a break in the top floor caused by
heavy tank that had been supported by the roof. At the same
ining building constructed with unprotected steel posts and beams
into a tangled mass of metal.

the Baltimore Fire. In the Baltimore fire there was but one
ncrete building of the three exposed to the fire, from which any fair
an be drawn. In one of the buildings, the concrete construction
destroyed, but this was probably due to the falling walls and the
her non-fire-proof parts. In a second building, the heavy rein-
ete floor of a banking-room came out practically unharmed; but it
osed to severe fire. The third structure was, however, exposed to
The contents of the building were destroyed and a large part of the
walls fell. The floors, five in number, were all of reinforced con-
ted on concrete columns, having replaced an old wooden-joist con-
test made after the fire showed that the floors were still strong
stain the loads for which they were designed, although the floor-
racked. The girders were cracked longitudinally near the lines of
ment, and the columns were spalled to such an extent as to expose
einforcement. It would have been difficult to restore the building
uld resist another such attack.†

the San Francisco Fire. The EFFECTS OF THE FIRE on concrete
in the conflagration immediately following the San Francisco earth-
6 are summed up in the following paragraph from the report of a
engineers that investigated the subject.

floors generally had hung ceilings, and, where thus protected, were
Where exposed, the concrete is in most cases destroyed, for instance,
Rialto, and the Aronson Buildings, and the Crocker Warehouse.
is dry, and while in many cases hard, yet all the water has been
nd it may be said to be destroyed, even if able to support weights.
gs of wood invariably burned, adding to the destruction. Sleepers
y burned. Surfaces of cement mortar fared much better, the lino-
g remaining practically intact."‡

ng the report, Mr. A. L. A. Himmelwright, who made a personal
the ruins, concludes that reinforced concrete is inferior as a fire-
truction to any form of steel construction with concrete floors and

ed reports are on file in the Bureau of Buildings, Borough of Manhattan,

ewell in his report on this building draws a different conclusion. See
ews, March 24, 1904, page 276.

Soc. C. E., March, 1907, page 330.

concrete column and girder-protection, but superior to steel construction w terra-cotta floor and terra-cotta column and girder-protection. "Where method was used, a very slight attack of fire was generally sufficient to cause rupture of the concrete underneath the reinforcing-metal, so that it fell aw exposing the metal. There were comparatively few buildings, however, in wh this method of construction was used."*

Thickness of Concrete Required. From a study of the tests and fires referred to, the fair conclusions as to the AMOUNT OF PROTECTION against would seem to be as follows: (1) In all columns and in large and import girders, trusses, or other supports, at least 2 in of concrete outside of all reinfo ments; (2) in girders and beams and in slabs of long spans, about 1½ in of c crete outside of all reinforcements; (3) in stair-work, floor-slabs of short sp and walls and partitions, from ¾ to 1 in of concrete outside of all reinforcem The provisions recommended in the Building Code of the National Boar Fire Underwriters are: "Steel reinforcement shall have a minimum p tection of concrete on all sides as follows: In columns and girders, 2 inc in beams and walls, 1½ in; and in floor slabs, 1 inch. The steel in footings walls and columns shall have a minimum protection of 4 inches of concre

"The minimum thickness of concrete surrounding and reinforcing meml one-quarter inch or less in diameter shall be one inch; and for members bea than one-quarter inch the minimum thickness of protecting concrete shal four diameters taking that diameter, in the event of bars of other than circ cross-section, which lies in the direction in which the thickness of the conc is measured; but no protecting concrete need be more than four inches t for bars of any size; and provided, further, that all columns and girders of r forced concrete shall have at least one inch of material on all exposed surf over and above that required for structural purposes; and all beams and f slabs shall have at least three-quarters inch of such surplus material for fire sisting purposes."

Other Forms of Protection for Reinforced Concrete. Because of effects produced by fire on reinforced concrete, as above described, and difficulty of restoring the construction where so affected, various suggesti have been made to protect the concrete construction with other materials. account of its excellent fire-resisting qualities (see page 817), CINDER CONCR naturally suggests itself. This material is out of the question where stren is required. But its use may be combined with that of STONE CONCRETE, placing a sufficient thickness for protective purposes on the outside of the r forcements in columns, below the neutral axis in beams and girders,† and on under surface of floor-slabs. Difficulties are likely to be encountered, howe in placing two kinds of concrete in the same mold, but these difficulties not insurmountable. Careful inspection is required to see that the poo material is not put in place of the stronger. One kind of concrete should fol the other immediately in order to secure a bond between the two. This s gestion, serving at the same time another purpose, was satisfactorily applied the column-protection in the Bush Terminal Warehouses in the Borough Brooklyn, New York. The steel-wire wrapping for the columns was prepa in sections 2 ft in height. Metal lath with about a ½-in mesh was placed outs the wrapping and secured to it. This was then placed in a cylindrical woo mold 2 ft in height and with a diameter 4 in larger than the wrapping, t forming an inner side of the mold. The space between the wrapping and t wooden mold was then filled with cinder concrete. When set and the m

* Trans. Am. Soc. C. E., 1907, Vol. LIX, page 305.
† Trans. Am. Soc. C. E., Vol. LVI, page 284.

he result was a hollow cylinder of cinder concrete, 2 in thick and 2 ith the column-wrapping attached to the inside These cylinders ne over the other in the building till the proper column-height was ch vertical rods as were wanted were put in, and the interior filled rete. Thus was produced a fire-proof, wrapped column, without d inconvenience of any column-molds in the building.

f fire-protection, advocated by the National Fire Proofing Company gh, Pa., is shown in Fig. 26. Here columns, beams, and girders are

Fig. 26. Tile Protection for Reinforced Concrete

ncased with HOLLOW-TILE BLOCKS. Being either laid in the molds hem, their rough and furrowed porous surfaces cause them to adhere e concrete They afford as efficient protection here as they do for s, and if destroyed the blocks can be replaced.

Aggregate on Fire-Resistance. Fire tests on full-size reinforced-lumns, conducted by Walter A. Hull at the Pittsburgh laboratories of f Standards, show that the nature of the aggregate plays an impor- resistance to fire. Silicious gravels appear to make unsatisfactory n the standpoint of fire-resistance. Limestone concretes are superior erials tested, including trap-rock and blast-furnace slags. Coatings aster, 1 in thick, and secured by a light, metal lath, protected the ffectively that the strength, after a four-hour fire test, was as much that of the unplastered columns after the test, and about 90% of olumn which had not been subjected to fire. Other forms of pro- ings investigated and proved effective, were a roofing-material of ent, sand, and asbestos, and cylindrical forms of cast gypsum, 3 in nd applied in a manner similar to those of cinder concrete described

ability of gravel concrete as fire-proof construction, was pointed out oolson as early as 1907* and appears to have been later confirmed

* Proc. Am. Soc. for Test. Mats., 1907.

by an actual fire in a reinforced-concrete warehouse at Far Rockaway, N. Y., 1916. From this the conclusion was drawn that "all concrete specificatio should contain a definite warning against the use of quartz gravel in concre liable to be exposed to high heat."*

6. Protection Against Corrosion in Reinforced-Concrete Constructio

Thickness of Reinforced Concrete. The THICKNESS OF CONCRETE requir for protection against fire has been found to be also ample for protection again CORROSION. It is well established that steel embedded in neat cement will n corrode. C. L. Norton of the Massachusetts Institute of Technology, Bosto Mass., draws the following conclusions from a series of experiments made 1902 and 1903.†

(1) Steel embedded in neat cement is secure against corrosion;

(2) Steel embedded in a dense concrete mixture is safe against corrosio

(3) To assure a thorough coating of the steel the concrete should be mixed we

(4) Porous concrete allows the admission of moisture and will not protect t steel thoroughly;

(5) A coating of rust is not a protection against further corrosion, as has be sometimes claimed.

In these experiments the steel was incased in concrete 1½ in thick on all side From this it would appear that

(6) The steel of reinforced concrete is secure against corrosion, provided it thoroughly embedded in concrete, and

(7) A slight coating of rust on the steel, where embedded, does no harn as the cement is strongly alkaline and will counteract the acidity of the ir oxide and prevent further corrosion.

"In practical design the most important question which arises is how f a concrete may be cracked (due to bending of beams) without exposing the ste to corrosive influences. In this respect it seems to the writer that the minu cracks which appear in the early states of the tests can have very little i fluence."‡ This means that within the safe working limits, there is no dang from corrosion on account of the fine cracks due to tension in beams and girde

Corrosion of Steel in Cinder Concrete. Cases are on record of serio CORROSION OF STEEL embedded in CINDER CONCRETE. In a report to the Stru tural Association of San Francisco, Cal.,§ the committee investigating the su ject states that in cinder concrete "the extent of the corrosion is great enou to seriously endanger the safety of the floors, and it is not probable that t floors would have supported their loads more than one to three years longer The committee recommended "that the Structural Association try to ame the present building law so as to exclude the use of cinder concrete in flo slabs or for fireproofing."

Mr. William H. Fox in his investigations ‖ on this same subject finds th "after about forty days' treatment, the specimens were broken, and the st carefully examined for corrosion. With but one exception, one or more of t three steel pieces in each specimen showed unmistakable signs of corrosi

* Report to National Board of Fire Underwriters, on the fire in question.

† Reports Nos. 4 and 9, Insurance Experiment Station of the Boston Manufactu Mutual Fire Insurance Company.

‡ Professor Turneaure in Trans. Am. Soc. for Test. Mats., Vol. IV, page 505.

§ Engineering News, Nov. 1, 1906, page 458.

‖ Engineering News, May 23, 1907, page 569.

' it made no difference how the concrete was mixed, wet or d
ed, whether the steam or water treatment was used, the
rust streaks and spots were found; the difference in the
eing imperceptible." He concludes that "to secure a d
ıder concrete, a thorough tamping is necessary. A ric
3 or one in which the proportion of cement to aggregat
ised in all cases. The greatest of care should be taken in
nd it may be necessary to resort to the seemingly impracti
he reinforcement with grout before placing in the concret
es of chemical and physical tests,* made by George Bor
ity of Nebraska, it was found that disintegration of cind
by the oxidation of iron and sulphur producing intern
ient cracking with occasional efflorescence of ferrous sulp
rom these tests, it was concluded that cinders with muc
e sulphur are likely to give unsatisfactory results, especia
e or porous material present; also that such material (ci
l if allowed to weather with occasional washing, until
ılphur have been washed and leached out of the cinc
l in these tests were from carefully screened steam coal
s showed considerable ferrous iron and sulphur as sulphi

iestion of the CORROSION OF STEEL in CINDER CONCRETI
:ludes: "There is one limitation to the whole question,
f getting the steel properly incased in concrete. Many
thing to do with concrete because of the difficulty in getti
: is especially true of cinder concrete, where the porou
has led to much dry concrete and many voids, and much
nothing in this whole subject has been more misunder
f cinder concrete. We usually hear that it contains mu
es corrosion. Sulphur might, if present, were it not for t
gly alkaline cement; but with that present the corrosi
ur of cinders in a sound Portland concrete is the veriest
f fact the ordinary cinders, classed as steam cinders, cor
nount of sulphur. There can be no question that cind
eat quantities of steel, but not because of its sulphur, l
too dry, through the action of the cinders in absorbin
ontained, therefore, voids; and secondly, because in a
contain oxide of iron which, when not coated over with
wet mixing, causes the rusting of any steel which it touch
ıd only one, mix wet and mix well. With this precauti
concrete quite as quickly as stone concrete in the ma

: Pabst Building in New York City, an eight-story st
was taken down after standing for about four years.
n the steel I beams in this case consisted of cinder c
uilt in segmental form.‡ The steelwork generally was f
t, though it should be remembered that all the stee
ıking all things into consideration it is probably safe to

[ndustrial and Engineering Chemistry, June, 1912.
9, Insurance Experiment Station, Boston Manufacturers]
ɔany.
stem, now obsolete.
Soc. C. E., Vol. L, page 297.

concrete, if care is taken to provide a proper mixture and careful and thoroug workmanship.

7. Erection of Reinforced-Concrete Construction

Forms for Reinforced Concrete. For the erection of reinforced concret it is generally necessary, first, to construct MOLDS or CENTERINGS for the c umns, floors, etc. Wood is the material used for this purpose. Sheet-met centering has been used with questionable success and economy. In the sel tion of the wood for the molds a clean grade of dressed pine should be use It should be thick enough to resist warping and to resist deflection between su ports. It must be coated on its surface with soap or some other satisfacto

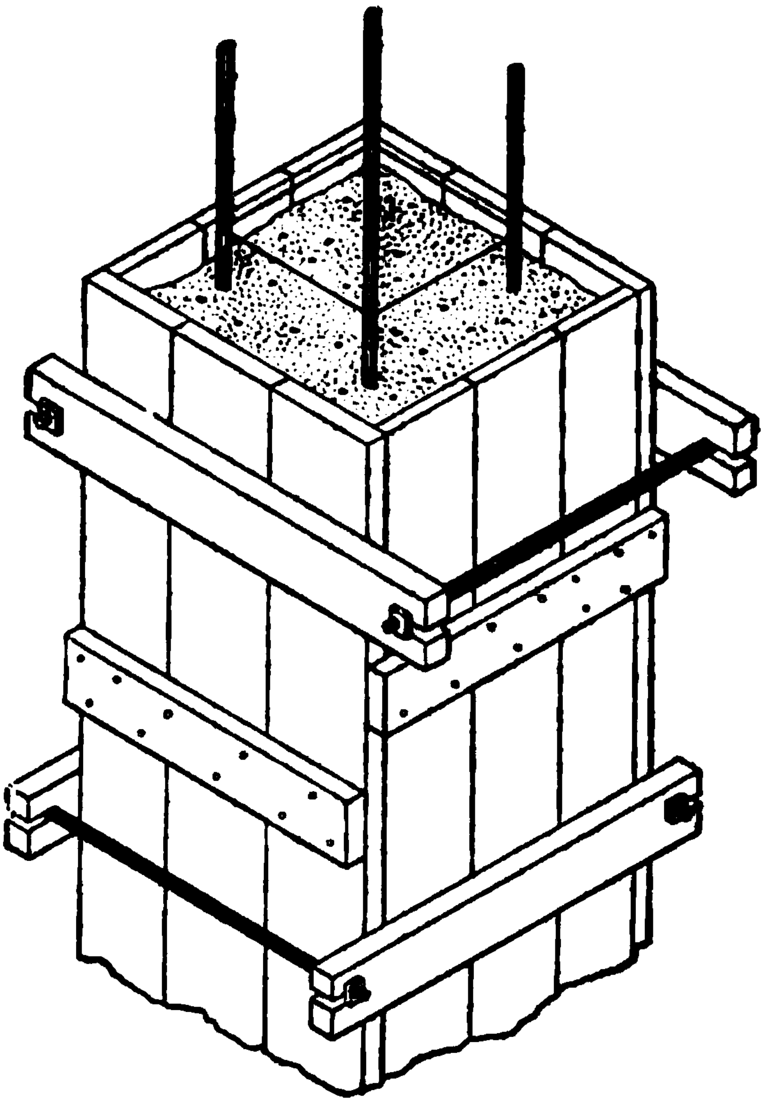


Fig. 27. Wooden Form for Reinforced-concrete Column

substance to prevent it from sticking to the concrete. The forms or molds m be erected carefully, the exact size of the proposed parts, and must be tru position and direction. For floor-molds, sufficient supports must be provid not only to carry safely the heavy wet concrete, but also such materials as liable to be placed on the floors up to the time when the concrete has set su ciently to carry such loads. The supports must have sufficient rigidity to prev deflection in the molds. The molds should be so constructed that they be easily removed when the concrete has set. Sharp corners should be avoi as much as possible, as the wood is liable to stick in them. Where there reentrant angles in the finished concrete-work, the molds should have bev edges, and at salient edges of the finished concrete-work, triangular strips sho be nailed in the corners of the molds to produce a beveled edge in the concr

: the spreading of the sides of the molds, cleats must be provided at intervals. In the case of beams and girders, these are gen- ed by nailing. In the case of columns and piers and often in walls, are so notched at the ends that long bolts with washers may be l them in place, as shown in Fig. 27. In removing the form the bolts l and the cleats and the rest of the form are ready to use again. In particularly in the construction of walls, the cleats are held in place ınning through the mold. These wires become embedded in the d in removing the molds they are cut and the portions in the con- owed to remain. The items of molds and centerings needed in the einforced-concrete buildings form a considerable part of the cost of ı. Economy in this respect can be affected in designing and planning he floor-panels throughout a building uniform in size and by repeat- s possible, such parts as piers, walls, etc. Successful attempts have › dispense with the erection of timber molds and centering by casting members of the construction on the ground and assembling and n in the same way that wood or steel columns, beams, and floors are ıd erected. (See page 953.)

Mixing. In all reinforced-concrete work the concrete should be ANICALLY. Satisfactory HAND-MIXING can be obtained and might be n very small jobs, where it would be uneconomical to set up a ER. But a much more uniform product will result from machine- nost types of mixers are mounted on wheels so as to be easily moved echanical mixers are either CONTINUOUS MIXERS or BATCH-MIXERS. ıous mixers the materials are fed sometimes by hand and sometimes , and the concrete issues continuously. The product, however, is he as uniform as that from the batch-mixer; for when the latter ınder constant supervision, whereas when the continuous mixer is hine is relied upon. Of the batch-mixers the ROTARY TYPE is the ıost general satisfaction. Among the efficient examples of this mentioned the mixers made by the Ransome Concrete Machinery ınellen, N. J., and the T. L. Smith Company, Chicago, Ill. They lifferent sizes and with capacities varying from about 10 to 60 cu yd

Concrete-Mixers. In CHARGING A CONCRETE-MIXER the materials , carefully measured, are dumped into the mixer and the machinery r completing a definite number of revolutions, sufficient to thor- ıe ingredients, the concrete is discharged into wheelbarrows or ıts for carrying it to the molds. Each batch should be completed is started. To obtain uniform results the number of revolutions on should be the same. It is not well to trust to the judgment of ırge of the machine, as to when the mixing has been thorough. ıstructed to count the revolutions each time. A good plan is to which rings when the fixed number of revolutions has been com- Code of the National Board of Fire Underwriters calls for "at ions," and the "speed of the mixer shall not exceed 20 revolutions

ete Mixture. The water is introduced during the process of ımount, also measured, should be such as to produce what is : MIXTURE, that is, a mixture that has the consistency of molasses ıdily flow around and thoroughly incase all steel to be embedded. ssary to vary the amount of water somewhat in placing a large te, as in walls, since the water generally works itself upward

through the successive layers. For TRANSPORTING THE CONCRETE from mixer to the mold, steel wheelbarrows, each holding about 2 cu ft, are gene employed. A larger vehicle, holding about 6 cu ft, is made by the Rans Concrete Machinery Company, Dunellen, N. J., and is found very economi in larger work. When the conditions will permit, concrete may also be dist uted by means of CHUTES, but care must be exercised to secure a consiste that will prevent the separation of the coarse aggregate from the mortar. transporting through the chutes may be done either by gravity or by c pressed air. Tests have shown that an excess of water tends to decrease strength of concrete, so that care must be taken not to use more water tha necessary to place the concrete properly.

Pouring the Concrete. Ideal conditions would obtain if the proce PLACING CONCRETE could be CONTINUOUS. This is not generally practic so it is important that the point at which work is stopped each day shall b selected and predetermined that the strength of the construction shall s least. In smaller buildings, with floor-areas not exceeding about 3 000 sq f should be possible to so arrange the progress of the work that each entire f construction may be placed in one day. In larger work it is necessary to off a certain area to be completed within the time of concreting for the Work should not leave off across important beams or girders, and the te rary stopping should be arranged for when the work is at the middle of or minor floor-beams. If any parts of floor-slabs are considered in the calc tions for the strength of the beams or girders, such parts must be concr at the same time and must be considered parts of such beams or gir Joints in columns should be made perpendicular to the axes of the colu and, as far as possible, at the lower side of girders. Columns should be allo to set for at least two hours before girders are cast on them, in order to pro for settlement and shrinkage.

Ramming the Concrete. As soon as the concrete has been poured int molds, and during the process of pouring, it should be continually RAMME secure complete filling of the molds, density in the finished product, and ough adhesion to the reinforcement. In wet concrete, such as is used for b ings, this ramming should be done with a flat steel spatula at the end of a ha long enough for comfortable manipulation. For column-work the hand lengthened out so as to reach to the bottom of the forms. Ordinary sp are sometimes employed, and where no special tools are provided, rammer sometimes made of 2 by 3-in scantlings, rounded off at the top end to a handle. Where a smooth surface is desired the spatula rammer shou used, particularly at the sides of the molds. The honeycombed appea that results from improper ramming is difficult to remedy afterward with surface of patches. After having been placed, the concrete should be damp by sprinkling it with a hose until it has thoroughly hardened. The ping of the forms with a hammer while the concrete is still plastic and bef has begun to set will cause it to flow more freely into place in intricate form around reinforcing-bars, especially when a dryer concrete, recommende recent investigators, is used. Tapping after it has started to set, however, to weaken the concrete.*

Removing the Forms from Reinforced Concrete. No fixed rule c given for the REMOVAL OF THE FORMS, as the time required for the setti concrete varies with the CONSISTENCY of the mixture, and the climatic and

* See "Effect of Vibrations, etc.," by Duff A. Abrams, Proc. Am. Concrete Vol. XV, 1919

Numerous failures of reinforced concrete have been attributed to ly removal of forms. In warm weather concrete will set more quickly d. The setting process may be somewhat accelerated after a day or noving the boards forming the sides of beams or girders and leaving ks on the underside and the props supporting them. In cold weather ible to warm the building during the setting process by means of s.

ish of Concrete Surfaces. The EXPOSED SURFACES of concrete ariously treated in attempts to produce a satisfactory appearance. pecial provision is made, the marks of the lumber used in the forms certain to show, and the lines of demarcation between successive clearly defined. To eliminate these lines, grooves are sometimes ormed, by tacking on the sides of the molds triangular or trapezoidal produce sunk joints in the wall, and give it an appearance resembling ne. The successive layers of concrete are in such cases stopped at so that the junction of the two layers is hidden. In some cases s purposely left rough and scratched like the scratch-coat in plaster- n stuccoed with a neat cement or a rich cement mortar. In this h there is always some danger that the stucco will flake off. The t comes from the mold, is sometimes hammer-dressed, or rather a special hammer. This hammer has an edge at right-angles to nd the edge is indented and made a series of points. A roughened produced which in time shows a uniform texture. Another method the forms as soon as the concrete is sufficiently hard and to rub the a plasterer's float or a block of carborundum, concrete, or stone, grout or fine sand with plenty of water between the float and the Brushing, also, may be resorted to, consisting in scrubbing the e still green, with a wire brush, and a mixture of one part of hydro- to six parts of water. A similar finish may be obtained by sand- the concrete has thoroughly hardened.

h of Reinforced-Concrete Floors. If the FLOOR-SURFACES are ered with a wooden flooring, a satisfactory finish may be obtained er the surface, before the concrete has had time to set thoroughly, sh from 1 to 1½ in thick, and troweling to make it smooth and finish is attempted after the concrete has set, the new and the old bably not bond; and there is always danger of flaking off unless the of considerable thickness.

Old and New Concrete. Various fluids and special cementitious e been put on the market for the purpose of BONDING NEW AND OLD RFACES. Whether or not these materials have any special merits, rally accepted that a good rich cement mortar will form sufficient two concrete surfaces, providing the surfaces are clean. If the PRESSIVE, the old surface of the concrete should be cleaned and surface may be roughened. Joints which are subject to TENSION ed with a 1 : 1½ or a 1 : 2 cement mortar before the new concrete ilding walls which must be water-tight, the structure should be THIC. And if it cannot, all dirt and LAITANCE should be removed, er of very rich mortar placed.

of Reinforced-Concrete Work. In all reinforced-concrete work ne importance to have competent and thorough INSPECTION or NCE. The inspector should be familiar with the nature and qual- ferent materials entering into the construction. He should have

a knowledge of the underlying principles of the design of reinforced-concre structures, so that he may realize the importance of carrying out all the detai and particularly of placing the reinforcement exactly as planned. He must l sufficiently alert and active to see that the work of the contractor is progressi properly; so that, for instance, work shall not have to be rebuilt because of err in the forms. The MATERIALS used in the construction, particularly the cemen should be tested as the work progresses. Cubes of the concrete as used shou be made up each day and at the end of seven days should be tested for com pression, and if necessary again at the age of twenty eight days. This recor will serve as a guide in the acceptance of the work, or in deciding on the nece sity for a load test of the finished structure. Under no circumstances, howeve should it replace or serve as an excuse to omit the testing of the cement up delivery or before acceptance. In addition to the details discussed in th chapter, details which require the attention of the inspector on the work, a fe others may be especially mentioned here:

(1) In JOINING NEW WORK with that which is already in, and which has beg to set, the surface must be thoroughly cleaned and wet. In stopping off work, is good practice where possible, to cast a groove in a surface that is to be join with another, so that when the work is afterward continued, a tongue-an groove junction is effected.

(2) All FORMS or MOLDS must be carefully cleaned out just before the concre is poured. The bottoms of the column-molds must be especially watched f this, as shavings, sawdust, and even blocks of wood are liable to fall into th unobserved. It is well to leave off a small piece of one side of the column-mo at the bottom, for purposes of observation and cleaning, and to close it up ju before pouring the concrete.

(3) Great care should be excerised in POURING and RAMMING concrete in de molds, such as for columns, walls, etc., in order to get the molds thorough filled at the bottom. In careless work it is not unusual to find in such pla very porous concrete, if not large pockets. This is particularly liable to occ when there is considerable reinforcing-steel in the construction.

(4) It should be remembered that concrete shrinks in setting. Hollow spa at the tops of columns are sometimes found to be due to this cause. As th are not always observable from the outside after the forms are removed, gr care should be exercised to guard against them. In pouring, therefore, t molds should be filled to overflowing to the top of deep molds.

(5) The exact position of the reinforcing-steel in the concrete is of such vi importance that particular mention is again made of it here. In loose-bar co struction the greatest care must be exercised, in the first place, to have the re forcement carefully placed, and then to avoid its being shifted out of positi by the pouring and the ramming of the concrete.

(6) The REINFORCING-STEEL of those systems in which the advantage of tached stirrups is claimed, is often, for convenience in shipping, sent with stirrups laid flat or close to the main bar. It is intended that in plac them on the job the stirrups shall be turned up to their proper positions. Unl carefully inspected, this is liable to be neglected.

(7) The use of a UNIT TYPE of construction (see page 922) practically obvia these two last-mentioned dangers, as the entire reinforcement comes framed gether, so that the relative positions of reinforcing-rods or bars cannot be chang and a glance will show whether the FRAME is complete or has been damaged, a when placed in the molds, whether it fits or not. In this type of construct the parts are all assembled in the shop from details carefully drawn and check in much the same way that steel beams, girders, columns, etc., are fabricated fr

p drawings. The work of the inspector or superintendent on the job
ı simplified, and hence the liability of error reduced to a minimum.

sts on Reinforced-Concrete Construction. Load tests on the
ıcture should only be resorted to when, all reasonable care having
ıd to obtain good results, some doubt still exists as to the results.
ıowever, should not be accepted in place of a strict compliance with
tions. The architect should know beforehand that his building is
igned and safe, and should employ, if necessary, an engineer. The
ıould understand at the outset that the structure has been designed
lefinite purposes and loads, and that the materials and details of
specified are not to be changed. If the contractor furnishes the
: sometimes does, a practice thoroughly condemned, the architect
ribe in his specifications that such design shall be checked and
an engineer appointed by him. A fair load to be applied in a test is
weight of the construction plus one-and-one-half times the working
he stresses in the construction are then equal to one-and-one-half
rking stresses assumed in designing. Under these conditions there
e any evidences of distress, and the deflections should not exceed
ı. The material used for the load test should be so selected and
when uniformly distributed, as required, it will not arch and assist
ive strength of the beam or floor. Pig iron is a very good material
ks are more generally available, but must often be piled very high
quired load, consuming much time and labor in making the test.
are used they should be set in vertical piles with spaces of 2 or 3 in
ı, thus avoiding all arching of the load.

CHAPTER XXV

REINFORCED-CONCRETE FACTORY AND MILL-CONSTRUCTION *

By

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General Principles. The problem involved in the proper design of a re forced-concrete factory or mill is a far more difficult one than might app from a superficial examination of the finished structure. This applies to bui ings constructed wholly or in part of reinforced concrete, and is due to the f that maximum economy and efficiency in production can only be obtai when the building is thoroughly adapted to a given occupancy and use. Laym and even some architects, look upon the factory as a mere workshop, consist of four walls with floors and roof. To them it seems an easy matter to loc the structure with reference to the lot or site and then supply it with stairwa elevators and kindred features. This, however, is not the case. Each indus uses processes peculiar to itself. The ease with which these processes can employed renders the profit-making more or less successful; hence it is nec sary to design the building to suit them. However, as the purpose here is explain what constitutes proper design, as applied to the reinforced-concr construction of a factory or mill-building, a typical case will serve to make cl the principles involved. This chapter, therefore, deals with such general ty as would seem to meet the needs of the greatest number of persons.

Walls, Floors and Roofs. Reinforced-concrete construction may be u for walls and floors, or for floors and roofs only, in the latter case substitut for reinforced-concrete walls some masonry construction such as brick or sto It is not always advisable to use reinforced concrete for walls. Circumstan very frequently arise in which it is more suitable and economical to use br walls or piers.

Types of Floor-Construction. The floor-construction may be divi into two general types, the BEAM-AND-SLAB type and the GIRDERLESS ty The beam-and-slab type may in turn be divided into varieties. For exampl may consist of beams supported by columns, with slabs spanning from beam beam. This arrangement corresponds to simple mill-construction in wood, wh the heavy timbers run across the building every 8 or 10 ft. The timbers r on the wall at one end and on a post at the other, with 3 or 4-in splined pla spanning from beam to beam. The earlier types of reinforced-concrete flo were patterned after this system. The next method was the introduction girders running from column to column, and the placing of the columns fart apart, say twice the distance common to the former system. The beams spaced as formerly. This may be called the BEAM-AND-GIRDER system. S another variation of the beam-and-slab type is the SQUARE-PANEL system,

* For Concrete in general and Mass-Concrete, see Chapter III, pages 240 to for Strength of Concrete without Reinforcement, Chapter V, pages 283 to 287; for Reinforced-Concrete Construction in General, see Chapter XXIV, the paragraph which, corresponding to the same details discussed here, should also be read. See, a Chapter XXIII, pages 817 and 844.

›eams are arranged along four sides of a square, a column being ıch of the four corners. The simplest type of reinforced-concrete for factories is some form of the beam-and-slab type with walls and forced concrete. The GIRDERLESS type consists of a heavy flat slab ı columns without the use of beams or girders. The column-head o form a large bearing-surface and the columns are spaced so as to bays as near as possible. A typical example is worked out at the hapter.

In general, as few columns as possible should be used to support ler that they may not interfere with the placing of machinery, and most economical use of the floor-space. From the standpoint of

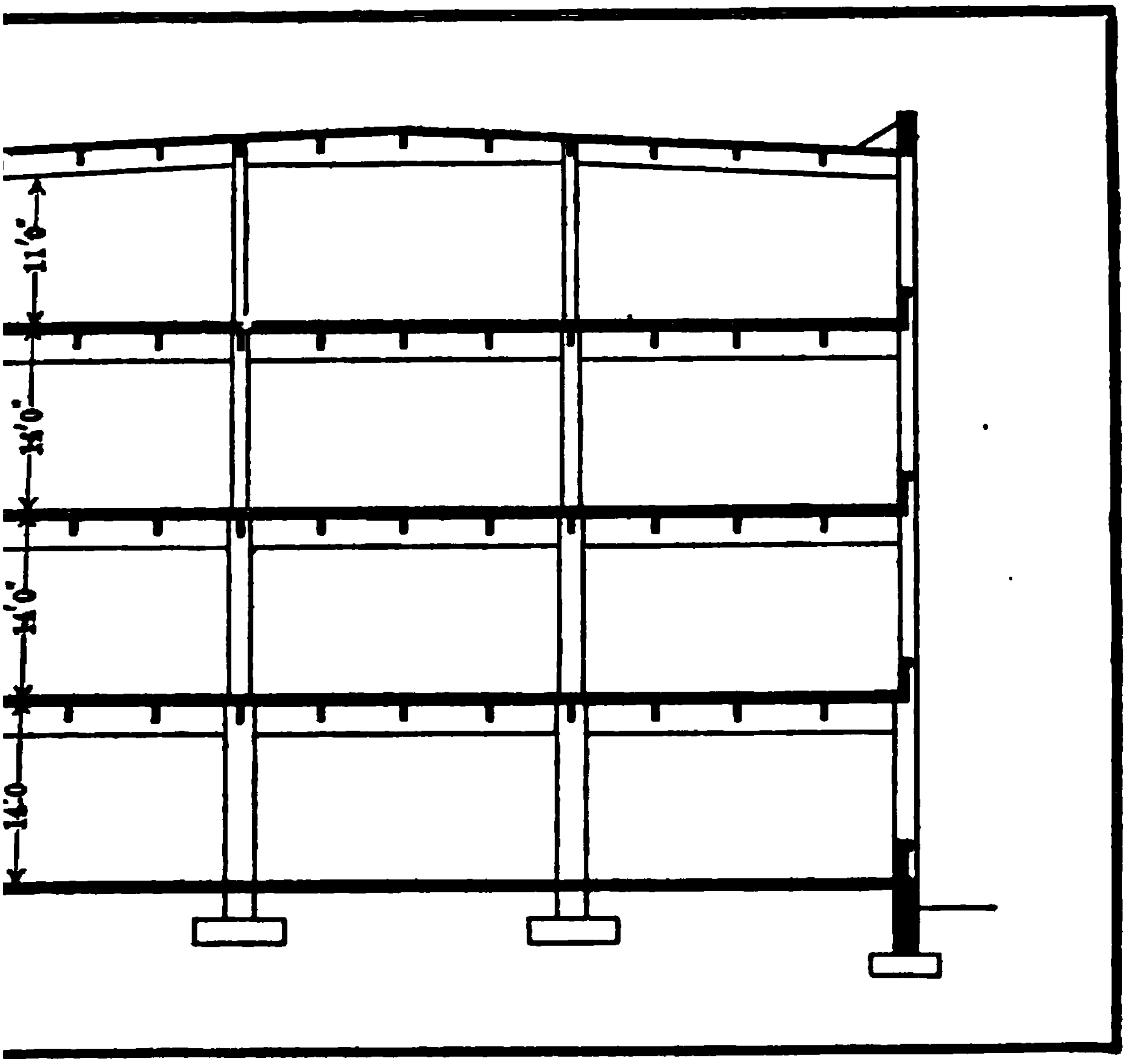


Fig. 1. Cross-section of Building

ıstruction, however, the use of one column to not more than or-space has been found to meet average requirements. This, ıot include construction of a special class. Adopting this, then, , and bearing in mind the fact that the nearer a building comes in plan the less is the total length of exterior wall required to area, it can be assumed that a four-story building 75 ft wide f columns, making three spans across the building, is a suitable ırposes. (See Fig. 1.)

g of a Building of this width, with story-heights of 14 ft, be ample for most purposes. There are always some parts of or which a strong light is not absolutely essential and which o aisles and to the storing of material in process of manufacture. : of the floor-space is generally used for this purpose, while the

machinery is placed nearer the windows where the light is best and where work is done. It is usually better, therefore, not to have a row of colu along the central axis of the building, unless it is definitely known that such arrangement will not interfere with the proper use of the floor-space. In building 75 ft wide, two rows of columns, with spans of 25 ft crosswise of structure, leave the central part of the floor-space free. Dividing 25 i 400 sq ft, the floor-space allowed for each column, gives 16 ft as the dista between columns, measuring lengthwise of the building

Bays. The reason in this instance for making the bays rectangular inst of square is that there would be another row of columns if a square bay w a maximum of 20 ft in either direction were assumed. This would be likely

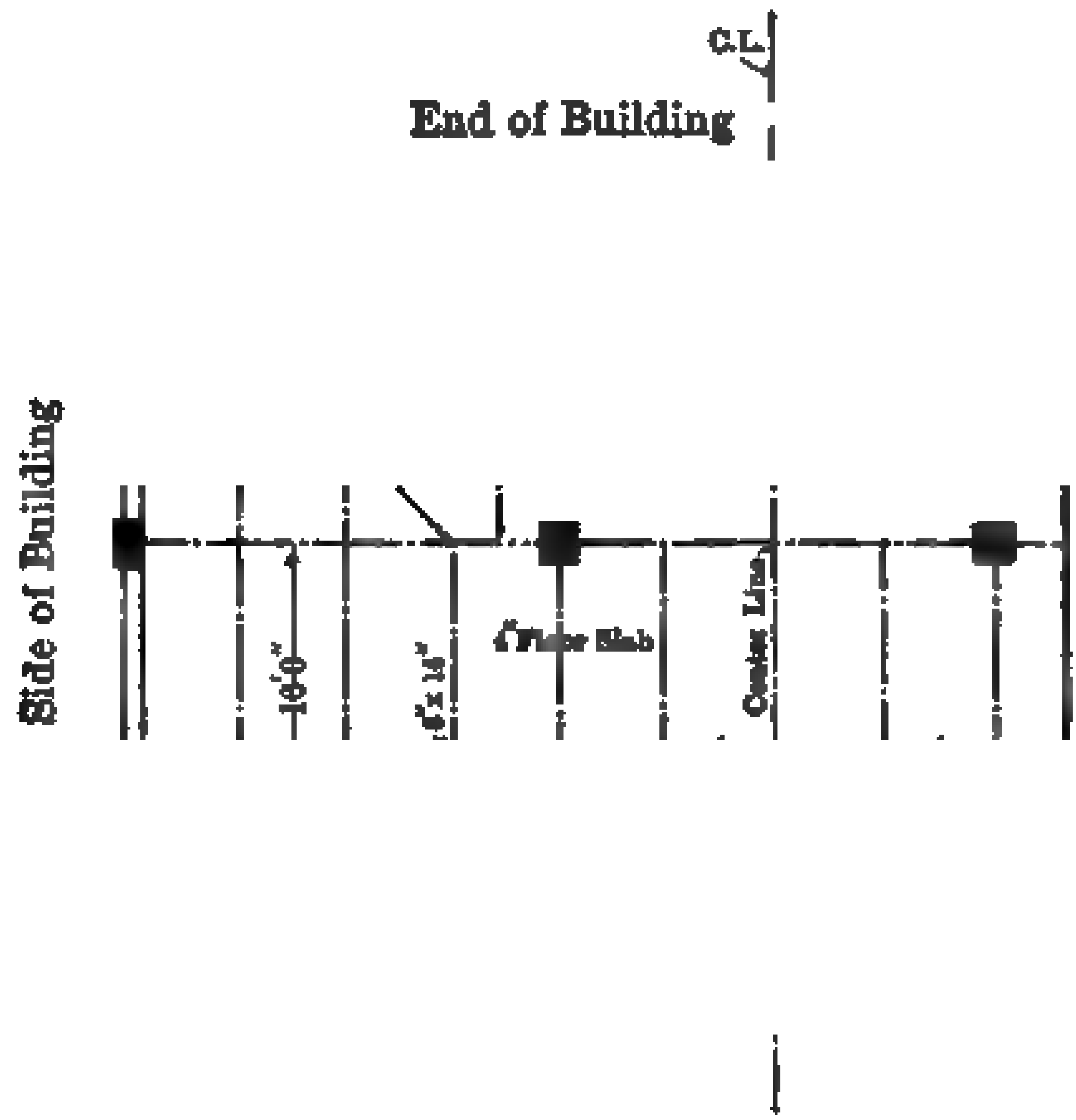


Fig. 2. Part Floor-plan of Building

interfere with the judicious placing of machinery and would result in a row columns along the central axis of the building. This is not considered g practice and should be avoided, except when there is to be only one row columns in the building.

Example of a Typical Bay The design of a typical bay of the size m tioned above, 25 by 16 ft, will now be considered. Referring to the illustrat (Figs. 1 and 2), it is seen that the windows occupy the major portion of the w area, the sill being set much lower than is usual in brick buildings. Th done to avoid the necessity of the construction of an extra-high spandrel be as the lintel over the windows below performs the double function of suppor the floor and forming a curtain wall. The head of the window is carried to the under side of the floor-slab to simplify the construction of the bot

and at the same time permit the window to extend to the ceiling, oducing the light at the highest possible point and allowing the rays r into the room. The first beam should be set as far back as possible side wall and windows, so that the angle of the direct rays of light early horizontal as practicable. It will be found best to have the run across the building, bearing on the walls and interior columns. s may be made as deep as economy of design suggests, as they run the light-rays and do not interfere with the lighting-scheme. Again, is relatively very economical. It also acts as a stiffener across the nension of the building, thus increasing the resistance to vibration oving machinery.

Floor-System. The various elements of the floor-system consist girders, beams and slabs. Each of these will be considered sepa- re load of 120 lb per sq ft is ample for light manufacturing purposes. oad prescribed by the Building Regulations of the City of Phila-

s. The spacing of the beams should be governed both by economy construction and the maximum distance a slab will span while load safely. It is impractical to make a slab less than 3 in thick. ght, with concrete weighing 150 lb per cu ft, is 37½ lb per sq ft. a 1-in cement finishing-coat, weighing 12½ lb per sq ft, to be laid te, the total live and dead load which the slab must carry, if it is 3 o lb + 50 lb = 170 lb per sq ft. Referring to the diagram of the einforced-concrete slabs (Fig. 18), calculated on a basis of the ent equaling $Wl/10$, no curve is found in the 3-in diagram for a o carry a load of 170 lb per sq ft. Some other slab must be used, arry the load.

Reinforcement. Referring to the diagram of the 4-in slab in llowing the 6-ft line until it intersects the horizontal line opposite q ft, it is found that a 4-in slab, reinforced with 0.195 sq in per lin n square bars per foot, will carry slightly more than is required for uestion. The total load, if the slab is 4 instead of 3 in thick, is q ft, and as the 187½-lb line is the nearest to this load, the 4-in d as above, is adopted. The reinforcing-rods are placed 1 in from the slab and are of sufficient length to extend over two spans and ch end; the joints are made over the beams and not in the space (Fig. 3).

s. The beams running from girder to girder are considered next span, center to center of girders, is 16 ft, and the distance apart ing an area of 100 sq ft carried by each beam. To the load per 182½ lb must be added the weight of the beam itself, which is 15 lb per sq ft of floor-area, making a total of 197½ lb per sq ft y the beam. This multiplied by the area, 100, equals 19 750 lb. oment caused by this load on the beam, based on the formula M = or partially restrained beams is the one generally used, is 379 200 b acts with the stem or beam to form a T beam and hence is as- e compression-flange of the girder; and as the slab is 4 in thick, the am and the amount of reinforcement can readily be found by refer- , which is the diagram of the strength of T beams having a 4-in m-depth in the diagram is the depth of the stem below the slab. n opposite the center of the space between 350 000 and 400 000 d side, the depth of beam that best suits the conditions can be t the bottom of the diagram is given the total area of steel to

be used in the reinforcing-rods. As the depth of a beam from the standpoin economical use of material should be about one-twelfth the span, a beam or 16 in deep is found to comply with this rule. Below the space where the l representing the 14-in depth of beam intersects the line representing the be ing moment, it is seen that the area of steel necessary is 1.8 sq in. Distribut

Fig. 8. Plan of One Bay, Showing Reinforcing

this over four bars, each bar should contain 0.45 sq in. The area of one 1⅛ square bar is 0.47 sq in, and hence a beam 14 in deep, reinforced with four 1⅛ square bars, is used. The width of the beam should be 6 in. A safe rule determine the width of the beam-stem is to allow 1½ in of concrete fireproo on the sides of the bars and arrange the bars in two rows, if the beams h three or more bars. The distance in a horizontal direction, center to cente bars, should be 2½ times the diameter, but in any case there should be a space between the bars horizontally, to permit the concrete to thoroughly in them.

Arrangement of the Bars. Assuming the bars to be twisted, the dista center to center, of the two bars is 1⅞ in. Adding to this the diameter of bars on their diagonal, which is about 1⅛ in, and 3 in for the fireproofing, sum is 6 in as the width of the beam required in this case (Fig. 6). It would perfectly practicable to arrange the four bars in one row across the bottom the beam; but the width would have to be 9⅝ in, which is wider than su requires. An additional objection to the latter arrangement is that it requ more concrete, thus adding to the dead weight of the construction. Th should be 2 in of concrete under the bottom of the rods for fireproofing.

Width of Beam. Of course, the width of the beam must be sufficient permit easy pouring of the concrete. Where wooden-box forms are used, i not good practice to make beams narrower than 6 in. If the beam is very d say 36 in, 6 in would be too narrow a width in which to place the steel a clean out the beam-forms. Practical considerations very frequently go the width of beams.

There should be in each beam and girder a sufficient number of le of at least 5/16-in round bars, bent U-shaped, run under the and extended up into the slab with an angle-bend 6 in long. If girder is short and excessively deep, 3/8-in round or heavier stirrups ed. The function of stirrups is to unite mechanically the slab to that perfect T-beam action will result, and also to assist in the DIAGONAL TENSION or SHEAR as it is commonly called. The num- s in a beam should be approximately one for each foot of the span, ter, but the spacing should be as stated below. Thus, a 16-ft have not less than sixteen stirrups, that is, eight on each side of e.

pacing for Distributed Loads. For beams with distributed rups are to be spaced so that the minimum distance between them in ordinary ne maximum nore than 36 iddle of the half of the be divided parts. The est the sup- contain ap- one-half the rups allotted he beam, or e total num- ddle division in one-sixth ber, and the to the cen- twelfth the as shown in distribution out evenly which comes this should

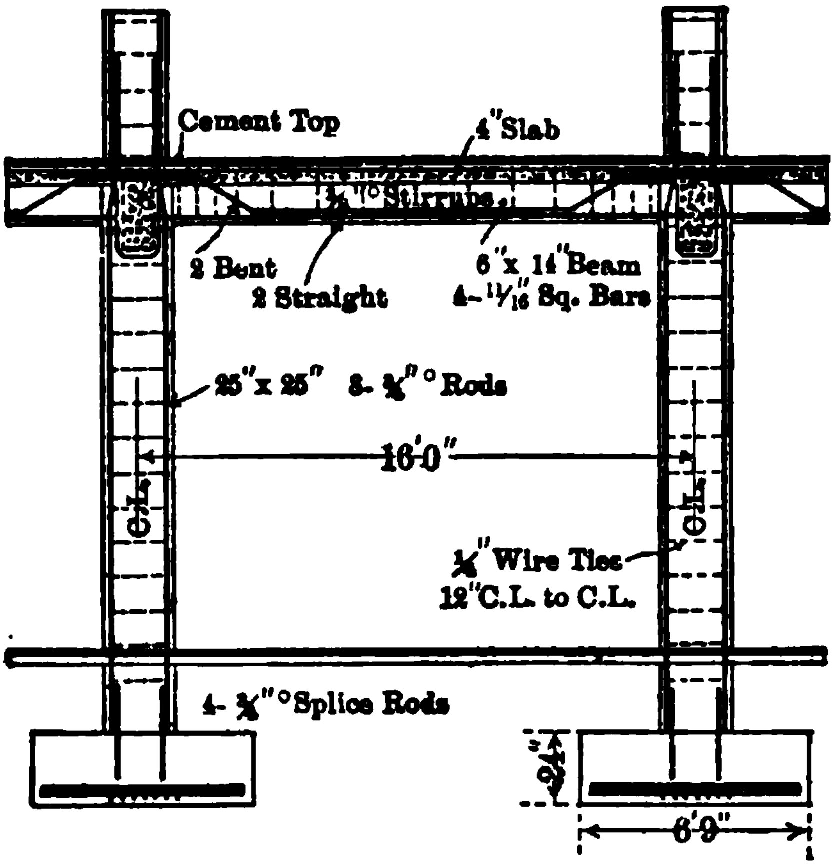


Fig. 4. Section Showing Elevation of Beam

acing for Concentrated Loads. When there are concentrated rups should be designed to suit the loading, but in any case, equal to about one-fifth the span from each end, the stirrups ced at least from 4 to 6 in on centers. A good rule to follow is ide of safety and to put in plenty of stirrups, if the determination umber is in doubt, as there should be a sufficient number of them part of the diagonal tension not safely resisted by the concrete.

gement of the Bars in the Beam is shown in Fig. 4. The are bent upwards near the supports to resist the negative bending a causes tension at the top of the beam near the supports. These tend into the next span at least 30 in to form a tie. As rein- e is of a monolithic character, it is necessary to introduce metal the concrete is subjected to tensile stresses. While it is not rovide as much steel at the top of the beam over the supports as restrained beams gives, if 50% of the area in the beam is carried

to the top and over the supports, as shown in the illustration, the beam will perfectly safe when calculated on a basis of M equaling $Wl/10$. In some citi beams must be calculated on the basis of $M = Wl/8$. Then it is only nec sary to have about one-fourth the number of the bars bent up near the support These bars, however, should extend at least 30 in beyond the center of t girder or column to tie the building together.

For Simple Beams with Uniformly Distributed Loads, all rods f 60% of the span should be straight and the truss-rods should bend up from t points so determined.

For Beams or Girders with Concentrated Loads, all bars are run straig as far as the concentrated loads extend. Beyond these loads, towards the su ports, one-half the number of bars may be bent up as above.

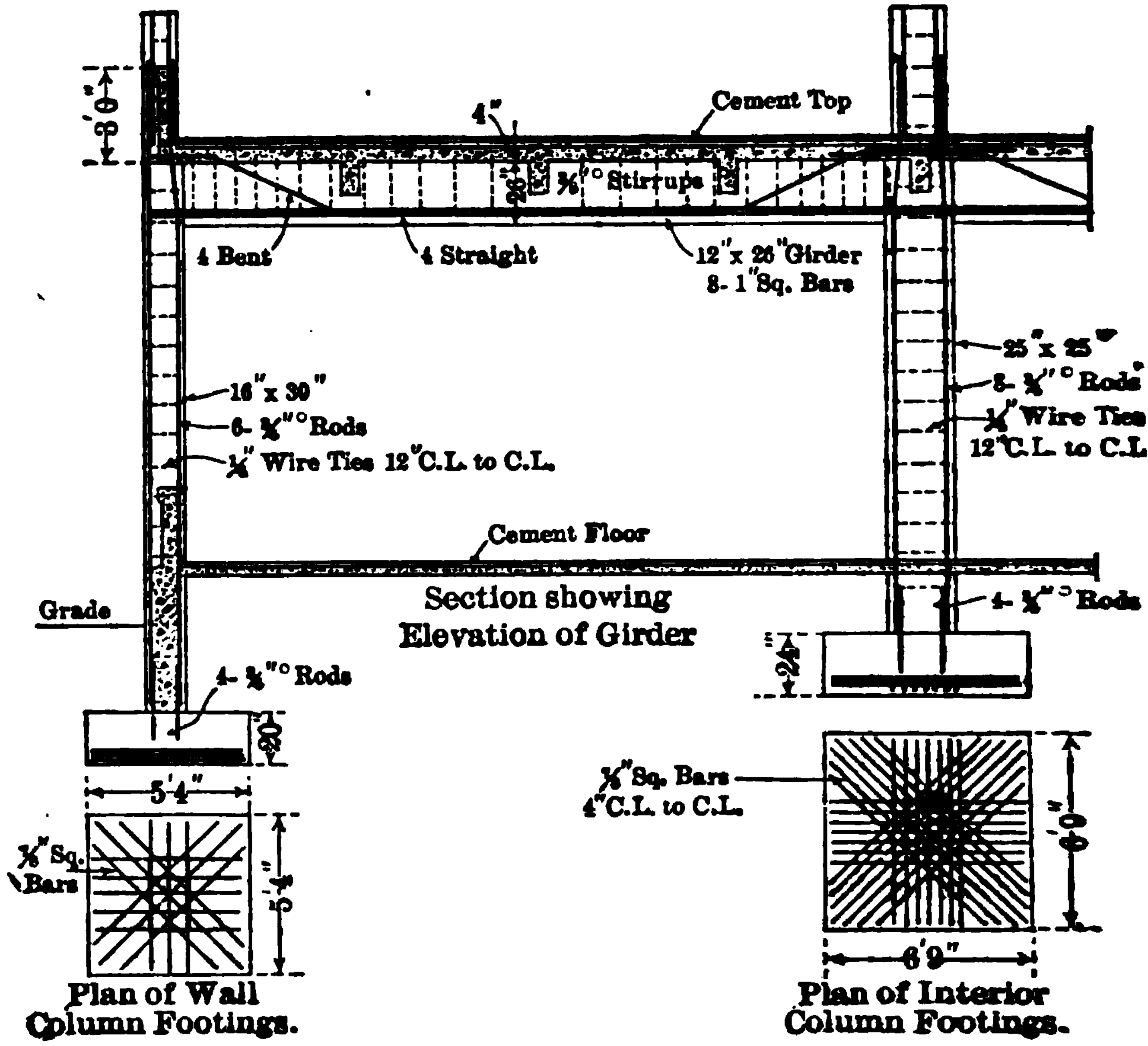


Fig. 5. Elevation of Girder and Plans of Column-Footings

The Girders. The girders running across the building are calculated on t basis of carrying their own weight as a uniformly distributed load and co centrated loads at the points where the beams frame into them. Referring the illustrations, Figs. 2 and 5, it will be noticed that there are three beams each side supported by the girder, the fourth beam being carried by the colum Each concentrated load equals the total load on the beams, or 19 750 lb. T weight of the girder can be assumed as 20 lb per sq ft of area carried, 20 × 4 = 8 000 lb. This acts as a distributed load. One-half the span of 25 ft, or 12

e bending moment at the middle of the girder from the con-
ibuted loads is

$$\times 150) - (19\,750 \times 75) + \frac{8\,000 \times 300}{8} = 3\,262\,500 \text{ in-lb}$$

s and Width of Girder. Referring again to Fig. 21, in ace opposite 3 300 000 and 3 250 000, the line of a 26-in deep tersect the vertical line representing 8 sq in of steel. Hence ars arranged in two horizontal rows are used. The width of n in order to have the proper distance between the bars and ive 1½ in of concrete fireproofing on the sides (Fig. 7).

section of Beam

Fig. 7. Cross-section of Girder

the Concrete Slab over the Girder is found by multiplying eel by the number on the line of the 26-in beam, which is t is used for any area of steel when the beam is 26 in deep, the ther beams are to be likewise used for their respective beams. girder, the width of the ⊤ beam is 8.7 × 8 = 69.6 in, or 34.8 the middle of the girder. The portion of the slab used at e girder or beam should not exceed on each side of the beam -thickness, nor one-third the span. In the case now being it is not exceeded. Similarly the width of the slab acting as ange of the 14-in beam is 1.8 × 12 = 21.6 in, twelve being the beams.

The next member to design is the lintel, or spandrel beam over 2, 5, and 8). This should be, for practical considerations, 6 in ttom of the lintel is flush with the bottom of the slab, the slab- o the lintel over the top of the lintel-rods. In addition to the tel there should be bars of the same size as the stirrup-bars, n apart and bent at right-angles, one leg extending up 12 in l the other 18 in out into the slab; or else the slab-bars should ding into the lintel 12 in. These make a perfect tie between l. The bottom of the lintel should be made with a rebate to of the window-frame. The load carried by the lintel is the b, the weight of the window and the dead weight of the lintel. floor-slab is 13½ ft (the clear span of the lintel) × 3 ft = 40½ the load per square foot on the floor-slab, or a total load from 371 lb. The total height of the lintel to the top of sill is 3 ft. k this makes the weight per lin ft 75 × 3 = 225 lb, the total el being 225 × 13½ = 3 038 lb. For the window 10 lb per sq ft area being 13½ × 11 ft, the height of the window, or in even

figures 149 sq ft, the weight is 149×10 lb = 1 490 lb. The total load on lintel, then, is 7 371 + 3 038 + 1 490 = 11 899 lb.

The Lintels Figured as Rectangular Beams. By referring to the paragr Explanation of Diagrams and Formulas, page 992, for the strength of rectang beams, it is seen that when reinforced with 0.5% of steel the safe load carried the beam is $W = wl = 48\frac{bd^2}{l}$. Hence, a 6 by (36 − 6)-in beam will carry $48\frac{6 \times}{13}$ = 15 552 lb. The depth 27 is used, as it is taken to the center of action the steel. This is more than the load upon the lintel and hence the linte safe. A reinforcement of 0.5% equals 0.005 of 162 sq in, the area of the concr or 0.81 sq in; and if two bars are u each must be of 0.4-sq-in sectional a Two ⅝-in square bars, each having area of 0.39 sq in, will be used. Th should be located 2 in from the botto and run straight. There should be t ⅝-in square bars near the top of lintel, running the full length, and fourt $^5/_{16}$-in stirrups, as shown in the illust tion (Fig. 8). The top bars take place of bent bars and also prev are vertical cracks which are liable to oc from shrinkage near the middle of lintel.

Fig. 8. Vertical Section, Showing Lintel

The Columns. Having establis the design of the floor-system, the dim sions of the wall piers, interior colum and footings are next determined. schedule of the loads on the inte columns will now be made.

The Load from the Roof. Assuming a live roof load of 30 lb per sq and 10 lb additional for accidental load from overhead shafting, the total l load is 40 lb per sq ft. The weight of the slab, if 3 in thick, which is as th as is usually required, is 37½ lb per sq ft. The beams and girders weigh anot 30 lb per sq ft (12 plus 18), making a total dead load of 70 lb, including t covering. Adding the live load of 40 lb to this gives 110 lb per sq ft as the to dead and live load.

The Load on the Fourth-Story Column, then, is 400 times 110 lb 44 000 lb, not counting the weight of the column itself. For practical reas no column should be made less than 10 by 10 in in cross-section. Allowi therefore, 500 lb per sq in unit stress on the concrete for columns, which is t unit stress allowed by the Philadelphia Building Bureau in reinforced-concr columns with vertical reinforcement, the carrying capacity of a 10 by 10 column is 100 times 500, or 50 000 lb, which is in excess of the load to be carri (See Table I.)

The Load on the Third-Story Column is the load from the one abo of 44 000 lb plus the load of one bay of the fourth floor, which is 217 lb × 4 = 86 800 lb, being the total dead and live load; or 86 800 + 44 000 = 130 800 lb, which must be added the weight of the column, which is assumed to be 300 lb lin ft. As it is about 11 ft long in the clear, the weight of the column is 3 300 which, added to 130 800 lb, equals 134 100 lb. The area of the cross-section a 16 by 16-in column is 256 sq in, which, at 500 lb per sq in, gives 128 000 lb

capacity. While this is 6 100 lb less than the load to be carried, of the required strength. It is customary to make a reducto be canied on the columns in proportion to the amount of the reduction being greater as the floor-area increases. Usuon of the LIVE LOAD per floor, with a maximum not exceeding m columns for high buildings, is considered good practice.

of Reinforced-Concrete Columns. Length, Fifteen Diameters

rtical bars. Safe working stress on concrete 500 lb per sq in, the being neglected in figuring the columns

Area	Total safe loads in lb	Size	Area	Total safe loads in lb
64	32 000	18×18	324	162 000
81	40 500	19×19	361	180 500
100	50 000	20×20	400	200 000
121	60 500	21×21	441	220 500
144	72 000	22×22	484	242 000
169	84 500	23×23	529	264 500
196	98 000	24×24	576	288 000
225	112 500	25×25	625	312 500
256	128 000	26×26	676	338 000
289	144 500	27×27	729	364 500

he Second-Story Column is 134 100 lb plus the load from the weight of the columns, all of which is assumed as being h-floor load and weight of column, or 90 100 lb, making the A 21 by 21-in column will carry 441 times 500 lb per sq in,

he First-Story Column is 224 200 lb plus the second-floor and the weight of the column, which, at 600 lb per lin ft, is of 317 600 lb. A 25 by 25-in column will carry 625 times or 312 500 lb, which is almost the required strength. The hen becomes

For the first story 25 × 25 in in cross-section.
For the second story 21 × 21 in in cross-section.
For the third story 16 × 16 in in cross-section.
For the fourth story 10 × 10 in in cross-section.

nent in the Columns should consist of eight ¾-in round er and four in the two upper stories, with ties of ¼-in round s shown in Fig. 9. It is the custom to use the same unit concrete columns up to 15 diameters, and not to use columns ls 15 diameters.

s. The schedule of all the wall piers is made by the e interior columns. The details of the calculations are not ults only being given. The size of the wall piers is deterhitectural effect desired and by practical considerations. the smallest face-dimension of the piers, this size should ll height of the building (Fig. 10). The reveal of the piers 5 in, and the spandrels should line up flush with the inside o doing they are not made extremely thick. Reinforced-

concrete spandrels may be 6 in thick and give good results. It is not wis make them thinner than this, on account of the difficulty of constructing th It is to be noticed, also, that the lintels or spandrel beams act as ties from wall pier to another. They should be of sufficient strength not only to carry vertical loads coming upon them, but also to act as braces to take up any vi

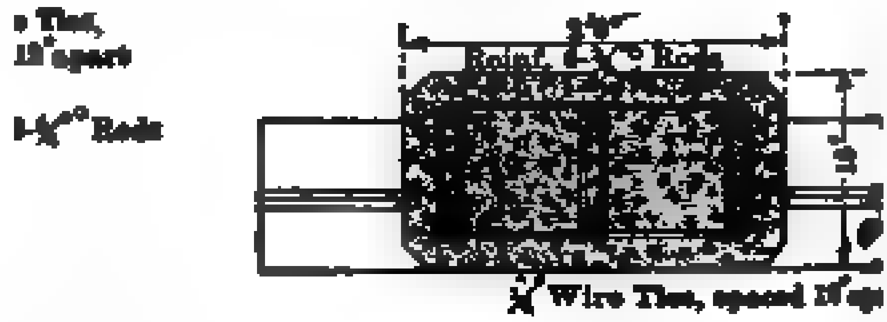


Fig. 9. Interior Column Fig. 10. First-story Wall Pier

tion in the direction of the length of the building; just as the deep cross-girde resist the vibration in the direction of the width of the building. Very fr quently the main girders are run lengthwise of the building, that is, spann the shortest distance, while the beams run across the building. Sometim this will make the construction more economical; but the reduced height the windows in the side walls due to the necessity of lowering the window-hea to permit the beams to be carried by a lintel running over them, is objectionabl as the light from the windows in this position is not as effective as when th are run up to the under side of the floor-slab or ceiling.

The wall-pier schedule, figured on the assumption above, becomes

For the first story 30 × 16 in in cross-section.
For the second story 30 × 12 in in cross-section.
For the third story 30 × 12 in in cross-section.
For the fourth story 30 × 12 in in cross-section.

It will be noticed that the piers in the three upper stories are of the sa dimensions. This is due to practical requirements, the reveal of the pier the spandrel being 6 in and the minimum spandrel-thickness 6 in. The p must be 12 in in order to be flush on the inside of the building.

Spread Foundations. The use of reinforced concrete for the footings a building results in economical construction when it is necessary to project t base or footing more than is customary or permissible without reinforcement some kind. In order to give sufficient information for the design of the found tions for the building under discussion in this chapter, as well as for other typ of construction met with in practice, several examples are worked out in t following pages. The simplest form of reinforced concrete SPREAD FOOTING shown in Fig. 5 and consists in considering the overhanging portions of t footings as CANTILEVER BEAMS. The footings of the interior columns are d signed as explained in the following paragraphs.

The Load on the Footing. The load on the footing is assumed to 317 000 lb and the safe bearing value of the soil 7 000 lb per sq ft. This requi a spread footing of 317 000 lb divided by 7 000, or 45 sq ft. The side of t square which comes the nearest to this area is 6 ft 9 in and its area is 45.5 sq f

The Design of the Footing. The footing is designed as follows: As ea square foot of footing sustains an upward pressure of 7 000 lb, the overhangi portion is treated as a CANTILEVER BEAM UNIFORMLY LOADED. The load directl

umn proper causes no bending, and this load is neglected in finding moment. The rods should be run as shown in Fig. 5, some diagone at right-angles to the sides, the first layer located 3 in from the footing. The size of the rods on the diagonal is now to be de- the others are to be made the same size. The longest length of diagonal cantilever is 4 ft, measured from the center of the column tion of the 1-ft-wide strip with the side of the square. The bend- n this strip is equal to the load on an area, outside of the column, 1 ft wide, or (3 × 7 000 = 21 000 lb) × 30 in = 630 000 in-lb, 30 in nce from axis of the column to center of gravity of the area.

e footing to be 24 in thick over-all, the CENTER OF ACTION of the out 5 in up from the bottom, making an EFFECTIVE DEPTH of 19 in. m for the steel is nine-tenths * of the depth when the stress in the lb per square inch, the resulting stress per square inch in the steel ess 16 000), is 16 000 × 0.9 = 14 400. As the bending moment lb, the number of square inches of steel necessary per foot

$$\frac{630\,000}{400 \times 19} = 2.34 \text{ sq in.}$$

This formula is for rectangular beams ng moment is given. (See Formula (1), page 992.) Spacing the nters requires three rods per foot, each requiring a cross-section in. As a 7/8-in square bar has a section-area of 0.76 sq in, this l. The bars in the layers at right-angles to the side are made the paced as above, so as to avoid complications in the construction It would be possible to space these farther apart, but this re- ecessary. (See Fig. 5.)

d on a column is such as to require a footing more than 2 ft thick, to slope the top of the footing, thus saving in the quantity of to provide a concrete PLINTH or BLOCK at the bottom of the of the footing so as to reduce the projection of the footing and a more economical design. If steel column-cores or hooped ertical reinforcements are used, a metal base-plate is necessary oting of sufficient size to limit the direct stress on the footing to

tions for the Outside Walls may be designed in either of as CONTINUOUS FOOTINGS such as are usual in ordinary construc- ly, as ISOLATED PIERS under the wall columns. In the first case o reinforce the footings and foundation-walls, as these act as ms loaded at each column, and must be made strong enough to ads from the columns uniformly over the entire length of the oundation-walls and footings can be treated as INVERTED CON- (Fig. 11), the upward reaction of the earth being considered a uted load on the beams, and the wall piers being considered as ng the beams, with the load on each pier as equal to the load on Fig. 12 shows the arrangement of the reinforcing-rods. Their d as explained in the following paragraph.

per running foot of the foundation is equal to the load from a e distance apart of the piers, omitting the weight of the spandrel ory windows, this load per running foot = 191 140 lb, the load 6 ft = 11 946 lb. As great refinements in calculations are not g-work of this kind, because of the advisability of large factors part of the building and the small reduction in cost due to any he strength of this continuous beam is calculated by the formula

), page 683; (4), 926; (6), 932; (1), 992; and Fig. 5, page 974.

$M = Wl/8$, assuming l to be the clear distance between the piers, or, in this ca
13 ft 6 in (Fig. 12). Therefore, $W = 13\frac{1}{2} \times 11\,946 = 161\,271$ lb and the be
ing moment $M = (161\,271 \times 162)/8 = 3\,265\,737$ in lb. As the size of the be
is determined by the thickness of the wall and its depth, all that is necessary is

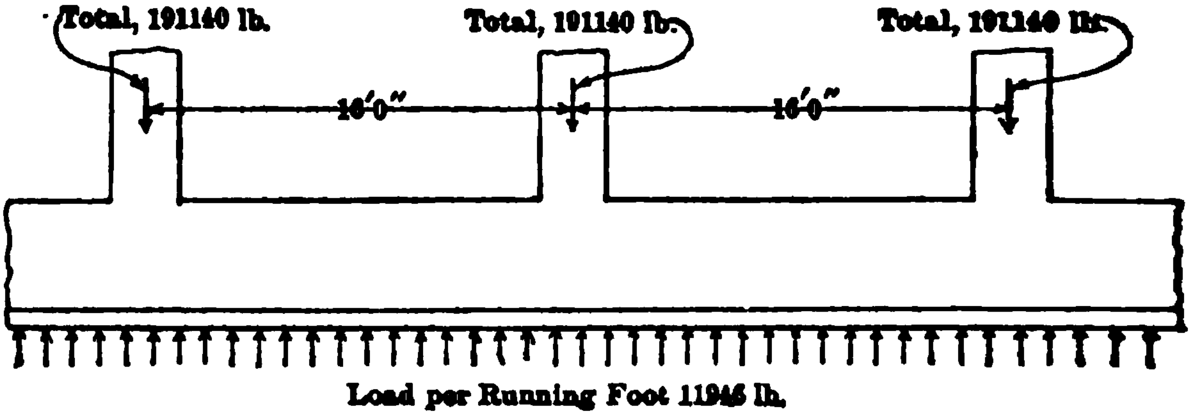


Fig. 11. Foundation-wall an Inverted Continuous Beam

find the AREA OF THE STEEL by referring to Formula (1), page 992, which gi
$A = \dfrac{M}{14\,400\,d}$, or $A = \dfrac{3\,265\,737}{14\,400 \times 52} = 4.3$ sq in, distributed in eight ¾-in squ
bars with a total area of cross-section of 4.48 in. These are in two layers, f
running straight and four bent as shown in Fig. 12. The top layer is placed 1

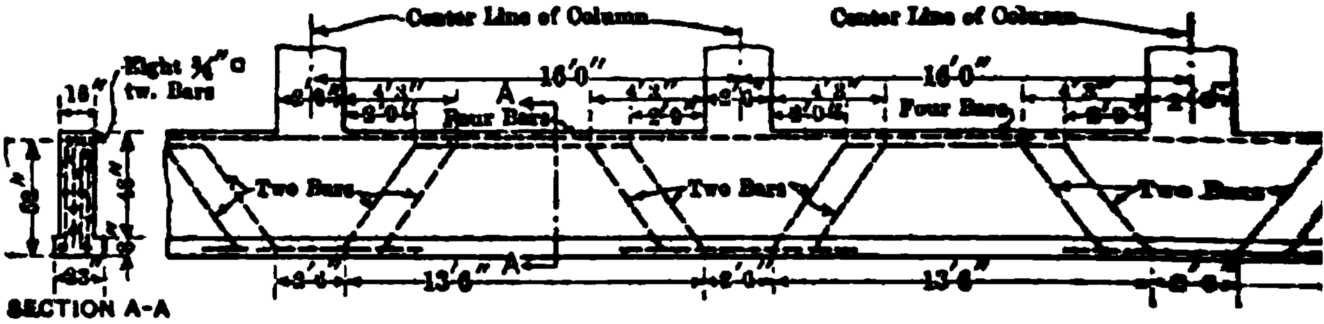


Fig. 12. Arrangement of Rods in Foundation-wall

from the top of the concrete. The footing is made wider than the wall to k
the load on the soil within the safe limit of 7 000 lb per sq ft. The width
determined as follows. As the column-spacing is 16 ft, center to center, 7 000 ×
= 112 000 lb, the load the foundation 1 ft wide and 16 ft long will carry; ha
to carry 212 120 lb (the load from the pier, plus 20 980 lb, the weight of
spandrel and footing), 212 120 is divided by 112 000, giving 1.9 ft for the wi
of the footing, or 1 ft 11 in, nearly.

Isolated Piers. In the second case, a SPREAD FOOTING is provided un
each wall column in the same manner as under the interior columns, but desig
for the lighter load. The foundation or spandrel wall is not made as heavy
in the first case, as it carries no load except its own weight and the wall or wind
above it. (See Fig. 5.) WHERE THE SOIL IS BAD and of low carrying capac
the PIER-METHOD is found to make an economical foundation, especially whe
is necessary to use piling under the building, as the piles can be grouped un
the piers and columns, and capped with reinforced concrete. The foundatio
spandrel walls, properly reinforced, can be carried from pile-cap to pile-cap
they do not depend on the soil directly under them to sustain the load.

Combined Column-Footings. It very frequently happens that a bui
ing is to be built adjacent to and abutting on a property-line, and as
foundations must not encroach upon the adjacent property the columns mus

the footings. In order to secure uniform soil-pressure it is
ombine an interior with an exterior column-footing so as to
uniformly from the two columns to the soil below. Some-
to combine the footings of more than two columns. Fig. 13
n actual construction and may be regarded as typical. The
mns in this case are almost identical, one being 700 000 lb

Fig. 13. Combined Column-footing

lb, so that the shape of the combined footing in plan can
he center of gravity of the two loads is practically at the
When one column is more heavily loaded than the other,
of the loads is no longer at the middle of the span, but
ence it is necessary to make the combined footing TRAPE-
. the center of gravity of the trapezoid will coincide with
e resultant of the loads from the columns.
ations for the design of this footing are the actual ones
good example of the necessity of assuming certain sizes at
al calculations may change. The WIDTH OF THE FOUNDA-
by the LOAD-LIMIT on the soil, which in this case is not to
ft, and the size of the column-base being known, we may
he BENDING MOMENT in the footing. We assume an area
, giving a soil-pressure of 1 490 000 lb ÷ 224 sq ft = 6 650
7 = 46 550 lb per running foot. The point of maximum
ere the vertical shear is zero and is determined by the
o = 15 ft. Also, 15 ft − 1.05 ft = 13.95 ft. Hence

o × 13.95) = 9 765 000 ft lb] − [(46 550 × 15 × 7½)
5 ft lb] = (4 528 125 × 12) = 54 337 500 in lb

The 1.05 ft is one-half the column-width, 2 ft 1 in.

We may determine the DEPTH OF THE FOUNDATION by assuming a c sectional area of the reinforcing-steel and solving in Formula (1), page 99 the depth. For practical considerations square bars larger than 1¼ in sq should not be used; hence by trial we find that thirty 1¼-in square bars w section-area of 46.8 sq in, placed in two rows in the top part of the founda will space out just right for a width of beam of 64 in, which is 6 in wider the 58-in dimension of Column No. 1. The depth then by this formula i

$$d = \frac{M}{14\,400\,A}, \text{ or } d = \frac{54\,337\,500}{14\,400 \times 46.8} = 80 \text{ in},$$

the depth from the center of the steel to the bottom of the concrete. There 80 + 4 = 84 in, the total depth of the foundation.

The WIDTH OF THE FOOTING AT THE BASE must be increased to keep the

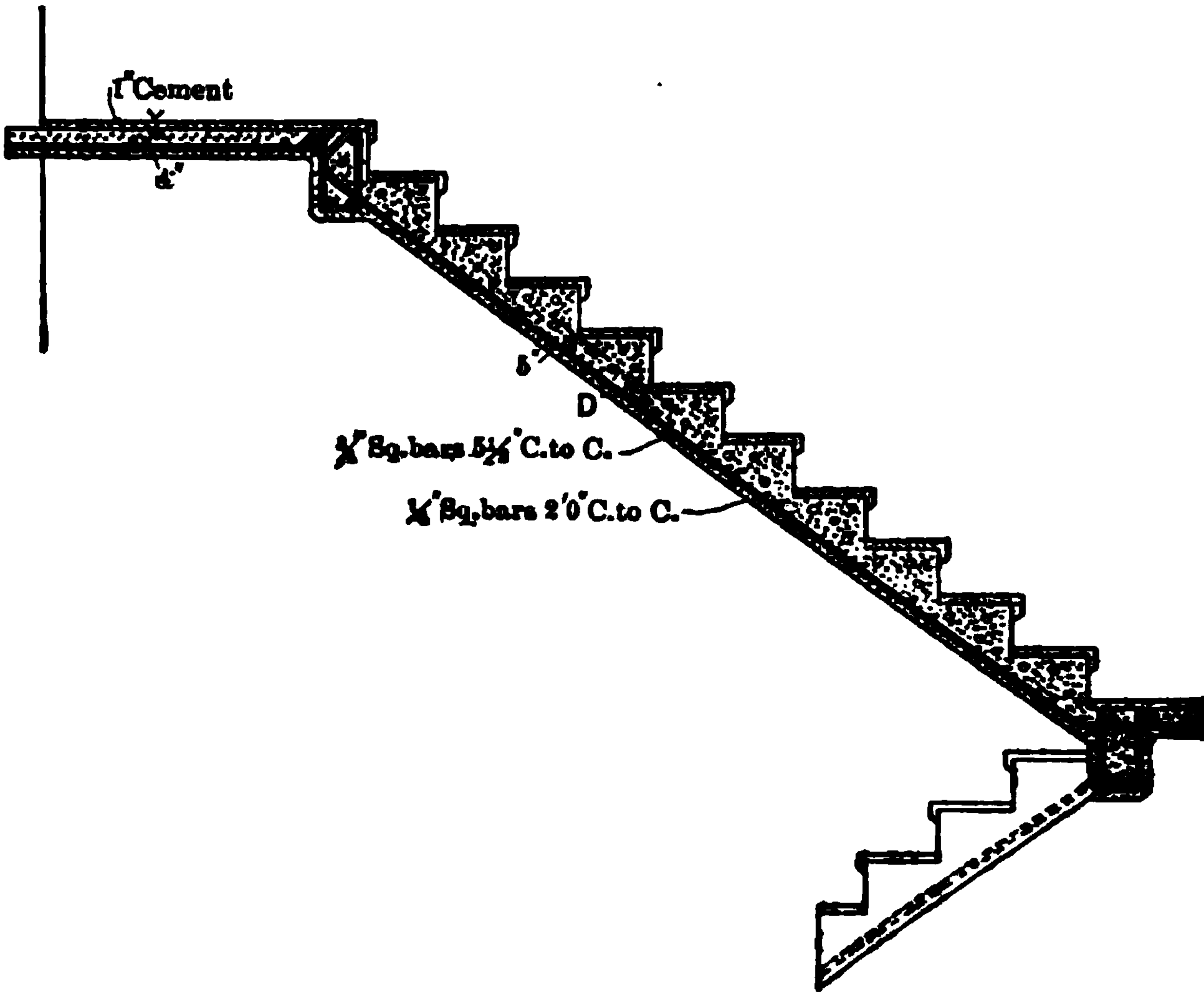


Fig. 14. Section through Flight of Reinforced-concrete Stairs

stress on the concrete in compression within the allowable stress, 600 lb per As the total horizontal compression in the beam must equal the total tens order to satisfy the requirements for equilibrium, we have total tension = lb per sq in × 46.8 sq in = 748 800 lb. From Table V, page 930, Chapter X the depth of the AREA OF THE CONCRETE IN COMPRESSION is equal to 0.3 = 24.8 in. The width is found by dividing 748 800 by (300 × 24.8 = the resistance of the concrete per inch in width of the beam, which gives 89 the width of the concrete at the bottom of the footing, 300 lb being the av unit stress on the area of the concrete in compression, since the stress act

600 lb on the outside upper surface of the concrete to zero at the

s. The ease with which stairs can be built of reinforced concrete has
eral adoption for this purpose in reinforced-concrete buildings. As
erally enclosed in stair-towers or shafts, their construction usually
n of the double run or half-pace type (Fig. 14). This reduces the
run so that the construction does not become too heavy. Each run
nsidered as an inclined beam and is so figured, being supported at
bottom on the stair-landing header-beam. The rods are placed
om of the slab and run continuously from top to bottom. The
beam is considered to be equal to the distance from the soffit of
he corner formed by the tread and rise, as shown by the letter D
he landings are figured the same as floor-slabs. Their supporting
culated to carry the load coming upon them from the landing and
r stair-run, which starts from the landing-beam. The lower stair-
p under the landing-beam, acts as an inclined strut and supports
beam. Hence the span of the landing-beam is equal to the distance
of the stair-tower or
side edge of the stair-
w, and is a little more
he width of the stair-
nakes the design of
rete stairs very eco-
pages 900 and 947.)

Stair-Design. It is
ach of the runs is 4 ft
t the maximum live
come upon the stairs
one person, weighing
h 2 ft of step, or 75
step. With steps 4
e load is 300 lb per
steps, 10 times 300
run for the live load.
s approximately 400
00 lb for the run. This makes a total load on the inclined beam
he span in calculating inclined beams is taken at the horizontal
n supports; hence in our example the span is 8 ft 9 in. The
ing moment, therefore, is $\frac{7\,000 \times 105}{10} = 73\,500$ in-lb, figuring
lly restrained. Assuming the thickness of the slab to be 5 in, the
s 4 in, and the area of steel for a 4-ft width for this depth and
as above is $\frac{73\,500}{4 \times 14\,400} = 1.3$ sq in, or 0.32 sq in for a 1-ft
⅝-in round bars, 4 in on centers, or ½-in round bars 7 in on
be used. (See page 1514) It is customary, also, to run
s, spaced 2 ft on centers, at right-angles to the main rods, as
It is also customary to run the rods which reinforce the run
n the wall-edge of platform at the top to the wall-edge at the
the rods to make them come in the bottom of the landing-
their reinforcement. The treads should be finished with a
of cement and grits; and the risers can be brushed smooth

Fig. 14. Detail of Reinforced-concrete Steps

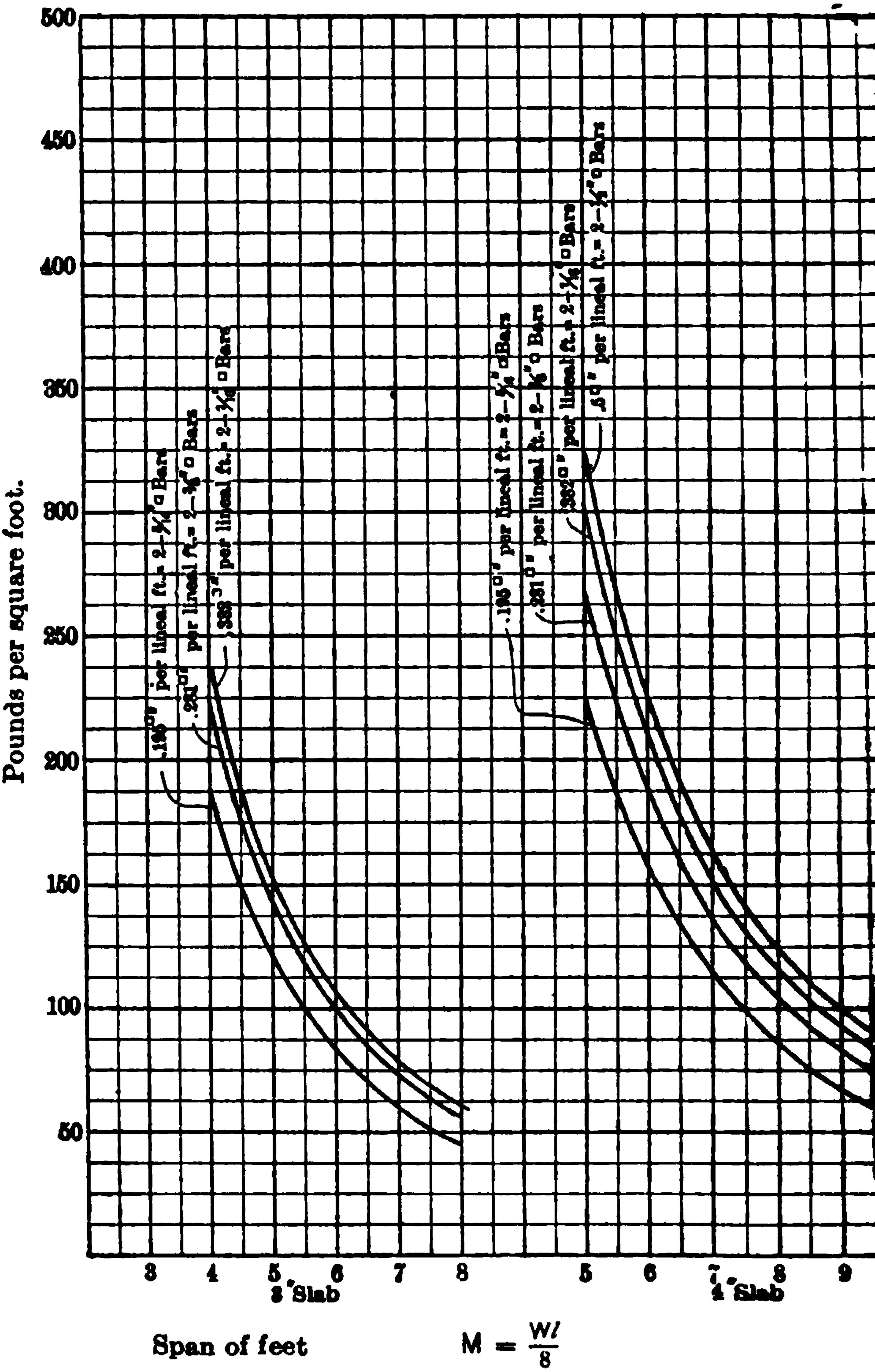


Fig. 16. Diagram for Strength of Reinforced-concrete Slabs

when their forms are removed. The riser-forms should be removed as as the concrete has set sufficiently to hold its shape, so that the top of each or tread can be incorporated into the concrete. Top-surfacing applied the concrete has set hard is very likely to become loose and break off. A good form of step is shown in the detail, Fig. 15. When the stair-runs

not be carried, at bottom and top where the steps start and eams, a reinforced-concrete beam, forming an outside string, l the stair-reinforcement run parallel with the risers from the The beam forming the string can be made any convenient and reinforced to suit the load.

8 9 10 11 6 7 8 9 10 11 12 13

lab 6 Slab

$M = \frac{Wl}{8}$

st

Diagram for Strength of Reinforced-concrete Slabs

agrams and Formulas. Figs. 16, 17, 18 and 19 are to reinforced-concrete slabs. These diagrams are plotted e in accordance with the 1907 Regulations of the Phila- ilding Inspection, which permit a unit compressive stress the concrete and a tension of 16 000 lb per sq in in the ie moduli of elasticity of steel to concrete equal to 12.

resses give a factor of safety of 4, based on the ultimate strength
s and have been found to give results in practice which are co
fety and economical construction, the concrete being a 1 : 2 :

3 4 5 6 7 8 5 6 7 8 9

6" Slab 8" Slab

Span in feet $M = \frac{Wl}{10}$

Fig. 18. Diagram for Strength of Reinforced-concrete Slabs

nd the aggregate a good hard stone. The building laws of
lly specify the allowable unit stresses to be used in designing rei
structures, and when they differ from those used in the calcu

to be made in the results obtained when using the diagrams. has the option of choosing his own method of calculating, used with absolute safety.

d 23 are diagrams of the strength of T beams. The cal- grams are based on the same unit stresses as above; but

$M = \frac{Wl}{18}$ # Slab

gram for Strength of Reinforced-concrete Slabs

beam is taken as the distance from the center of action r of the concrete slab and not to a point one-third the m the top. The beam-depths in the diagrams are the w the slab. The width of the slab in compression is e area of the steel by the constant given in the dia- fling depth of beams.

Bending moments in inch-pounds.

Area, A, of steel, square inches.

Fig. 20. Diagram for Strength of T Beams

DIAGRAM OF STRENGTH OF T BEAMS

Area, A, of steel, square inches.

. 21. Diagram for Strength of T Beams

DIAGRAM OF STRENGTH OF T BEAMS

Bending moments in inch-pounds.

Area, A, of steel, square inches.

Fig. 22. Diagram for Strength of T Beams

DIAGRAM OF STRENGTH OF T BEAMS

Area, A, of steel, square inches.

g. 23. Diagram for Strength of T Beams

The following formulas are for the strength of rectangular beams or slal based on various percentages of steel, the beams being considered to be as simp supported at the ends. They are calculated in accordance with the Philadelph requirements, and can be used in investigating the strength of rectangul beams and slabs without obtaining the bending moment. They are very co venient in checking up a design already made, or in establishing the area of t steel reinforcement when the size of the concrete beam or slab is fixed, as show by the example given.

w = load in pounds per running foot;
b = breadth of beam in inches;
d = depth to center of action of steel in inches;
l = span in feet;
p = percentage of steel to area of concrete above center of steel to top of bea

When $p = 0.5\%$ then $w = 48\dfrac{bd^2}{l^2}$

$$p = 0.6\% \qquad w = 56\frac{bd^2}{l^2}$$

$$p = 0.7\% \qquad w = 59\frac{bd^2}{l^2}$$

$$p = 0.8\% \qquad w = 62\frac{bd^2}{l^2}$$

$$p = 0.9\% \qquad w = 64.5\frac{bd^2}{l^2}$$

$$p = 1\% \qquad w = 67\frac{bd^2}{l^2}$$

Example. Find the total load per square foot that can be carried by a 4 slab, with a 5-ft clear span, reinforced with 0.8% of steel per running foot.

Solution.

$$w = 62 \times \frac{12 \times 3^2}{5^2} = 62 \times \frac{108}{25} = 266.6 \text{ lb}$$

From this must be deducted the weight of slab and floor-finish to obtain live load. If finished with 1-in cement top coat laid directly on the conc the total dead weight is 62½ lb, which, deducted from 266.6 lb, leaves 204.1

Note. If the total load carried by the beam is desired, use l instead o in the formula. These formulas are based upon the stress in the concrete exceeding 600 lb per sq in and a tension in the steel of 16 000 lb per sq in, a ratio of the moduli of elasticity of the concrete and steel equal to 12.

Formula for the Resisting Moment of Rectangular or T Beams. T is Formula (6), page 932, Chapter XXIV, only in a different form, and is to used when the percentage of steel is not greater than 0.58 of 1%.

M = the maximum bending moment in inch-pounds;
d = the depth from the top of the beam to the center of action of the stee inches;
A = the area of the sum of the cross-sections of the steel bars in square inc

$$M = A \times 16\,000 \times 0.9\,d \text{ or } A = \frac{M}{14\,400\,d}$$

or $$d = \frac{M}{14\,400\,A} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; .$$

Given a bending moment of 217 728 in-lb and a depth (over all) in, to find the sectional area of steel necessary to make the resisting l to the bending moment.

$$A = \frac{M}{14\,400\,d} \text{ or } A = \frac{217\,728}{14\,400 \times 13\frac{1}{2}} = 1.12 \text{ sq in.}$$

ound bars of ¾-in diameter we have 0.56 sq in × 2, or 1.12 sq in. for fireproofing and ½ in to the center of the bars, the effective eam is reduced to 13½ in. For the width of the beam we can use age 931, Chapter XXIV, substituting for K the value correspond-t stresses and the ratio of the moduli of elasticity for the con- we have been using, namely, 600 and 16 000 lb per sq in for the nd 12 for the ratio. This value of K, from Table V, page 926, ', is 83.4 and $M = 83.4\,bd^2$. Transposing, we have

$$\frac{M}{3.4\,d^2}, \text{ or } b = \frac{217\,728}{83.4 \times (13\frac{1}{2})^2} = \frac{217\,728}{83.4 \times 182.2} = 14.3 \text{ in}$$

erefore will be 14½ in × 16 in in cross-section, reinforced with two s placed so that there will be 2½ in from the bottom of the beam As the width of this beam is excessive for the number of rods nomical. It would be better to design the beam with a T section dth to 6 in for the stem and making the top flange 14½ in wide = 4.18 in thick. The ratio of the distance of the neutral surface f the beam to the effective depth of the beam, for the values we is 0.31 (see Table V, page 926, Chapter XXIV), and in order to concrete in compression at the top of the beam to balance the the steel, the head or flange of the T must extend from the top e neutral surface.

loors.* In order to familiarize the student with the design of RS, an example is worked out, in which the area of a panel or bay 400 sq ft, the same as that of a typical bay in the factory-build- idered in this chapter. The column-spacing is made the same s, so that the panels are square, with a length of side of 20 ft. ng the various methods of computing the strength of flat, re- plates, we will use one under consideration by the Bureau of ion of Philadelphia.† This is a conservative method. It has orked out in all its details and applications and gives results safety and economical design. The following paragraphs set n and equations of this method as published by the Philadelphia ls it the DROP-CONSTRUCTION.

n, center to center of columns, of the longest of straight bands es.

nce or width, edge to edge, between capital-heads in inches dead and live load per square foot.

ce from the center of action of the concrete in compression to ter of the steel at the drop in inches.

ce from the center of action of the concrete in compression to ter of the steel at the center of the slab in inches.

* pter XXIV, pages 949 to 951. Flat-Slab Construction.

† rk, Chief of the Bureau of Building Inspection, Philadelphia, Pa., is orking out and perfecting the practical applications of this method.

If the drop-construction is not used, $d = d_1$.

Sufficient depth of slab is to be provided for shearing-stresses as well as bending-stresses.

Width of capital-head = not less than 2/10 L.

Width of drop = 38/100 L.

Width of bands = 45/100 L.

x = the area of section of steel over the capital-head.

x_1 = the area of section of steel in center of bay.

$-M$ = the bending moment at the edge of the capital-head.

$+M$ = the bending moment at the center of the span.

$$\text{The load carried by the straight band} = \frac{\text{total bay} - \text{capital-head}}{2} \times w$$

$$-M = \frac{\text{total bay} - \text{capital-head}}{2} \times \frac{wL_1}{12}$$

$$+M = \frac{\text{total bay} - \text{capital-head}}{2} \times \frac{wL_1}{24}$$

Width of concrete to resist compression at edge of capital-head = width drop.

Width of concrete to resist compression when negative moment = $-$

= width of band.

Width of concrete to resist compression at middle of span = width of ba

$$x = -\frac{M}{d \times 16\,000}$$

Place 66% of x in straight bands } over capital-head.
Place 43% of x in diagonal bands }

$$x_1 = +\frac{M}{d_1 \times 16000}$$

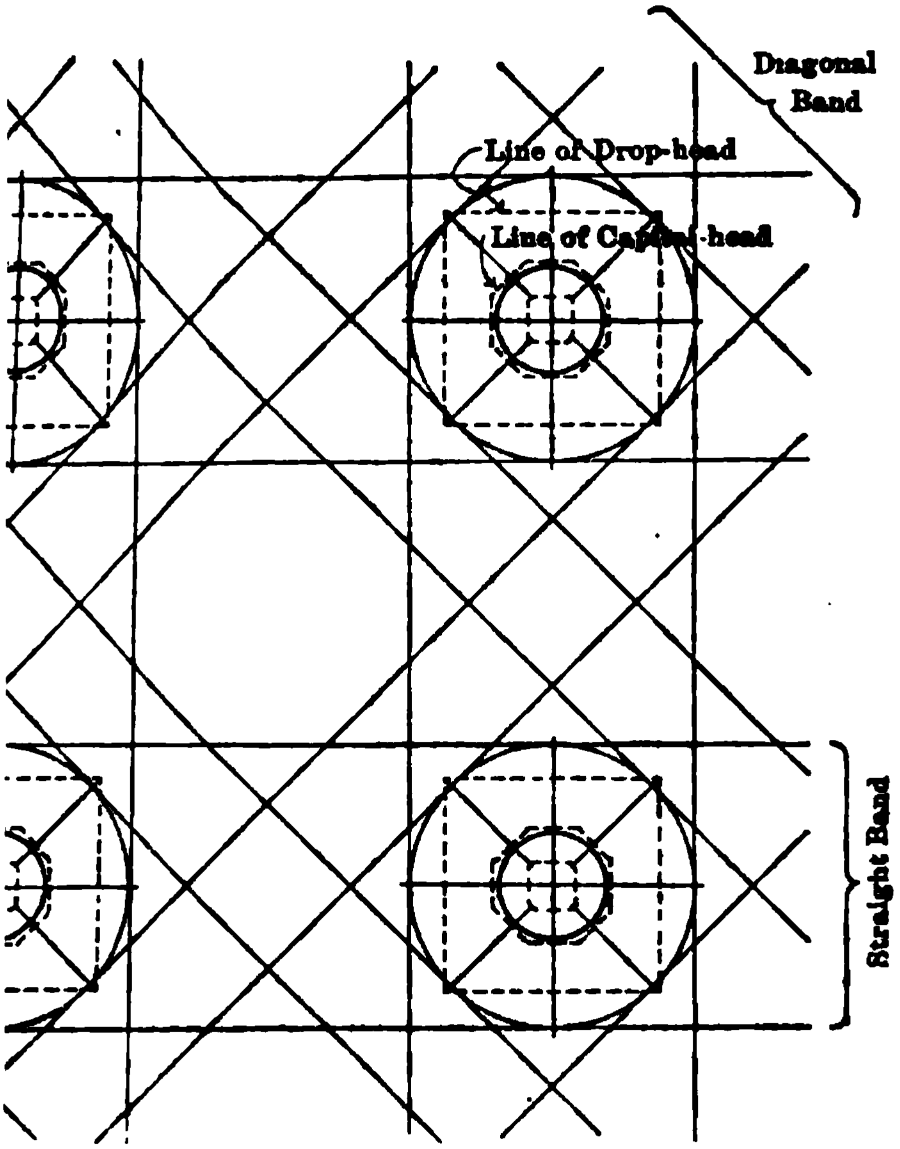
Place 66% of x_1 in straight bands } at middle of span.
Place 43% of x_1 in diagonal bands }

Add 20% of steel of $-M$ to provide for negative bending over straight ba

The drop equals the abacus outside of the capital-head, or the incre thickness of the concrete to obtain the necessary compression in the conc This is not generally necessary when the live load of the floor is light, say 1 per sq ft and the span is not excessive. To determine d and d_1 deduct from total thickness of the slab 1 in to the center of the steel when the rods are or less in diameter; if over ⅝ in deduct 1½ in; and multiply the result by The result will be the distance from the center of the steel to the center of a of the compressive stresses in the concrete.

The depth h is the distance from the top of the slab to the center of the and is used in finding the thickness of the slab. Applying the above form to the example considered, using a floor-load of 120 lb per sq ft as in the prev example, and assuming an average slab-thickness of 8 in with a 1-in top fi coat of cement, the dead load is 100 lb + 13 lb = 113 lb, which added to th load = 233 lb, total.

The arrangement of the bands is shown in plan, Fig. 24, the width being 4 or 45/100 the span of 20 ft, which is 9 ft. The diameter of the column-head is or 4 ft. The width of the drop is 38/100 L, or 7 ft 7 in.



4. Arrangement of Bands in Girderless Floor

e bay $= 20^2 = 400$ sq ft.
ital-head $= 4^2 = 16$ sq ft. Then, by the formula, the load
t bands $= \dfrac{400-16}{2} \times 233 = 44\,736$ lb

$$\frac{\ 736 \times L_1}{12} = \frac{44\,736 \times 16 \times 12}{12} = 715\,776 \text{ in-lb}$$

$$\frac{\ 736 \times L_1}{24} = \frac{44\,736 \times 16 \times 12}{24} = 357\,888 \text{ in-lb}$$

t diagram is shown in Fig. 25.
to find the thickness of the concrete at the drop. The
e depth of a beam when the bending moment, the width
owable stresses are given, is as follows, in which h equals
ab from the center of the steel to the top of the concrete:

$$\sqrt{\frac{M}{\ bS_c}} = \sqrt{\frac{2 \times 715\,776}{0.27 \times 91 \times 600}} = \sqrt{100} = 10 \text{ in}$$

the width of the drop and $S_c = 600$ lb per sq in. The
all, therefore, is $10 + 1 = 11$ in (Fig. 26).

lumn at the drop $= x = -\dfrac{M}{d \times 16\,000}$ in which $d = 0.9\,h$

$$\frac{715\,776}{9 \times 16\,000} = 4.9 \text{ in, or about } 5 \text{ in}$$

The straight band will have 66% of 5 or 3.3 sq in of steel. A 3/8 round rod h a cross-sectional area of 0.11 sq in. Therefore, there will be $\frac{3.3}{0.11} = 30$ bars ov the capital-head in the straight band. As the bars from the adjoining span ov lap the column-head, extending into the next span as far as the edge of the dr each straight band over the column will have 30/2 or fifteen bars. The diago

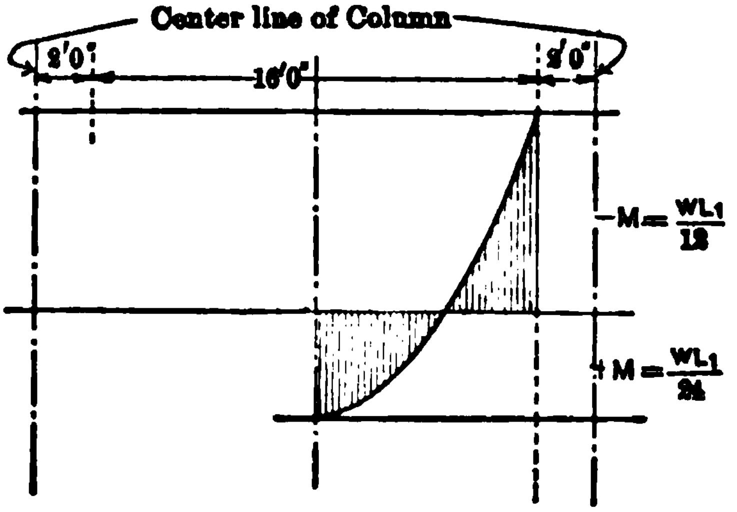


Fig. 25. Bending-moment Diagram for Girderless Floor

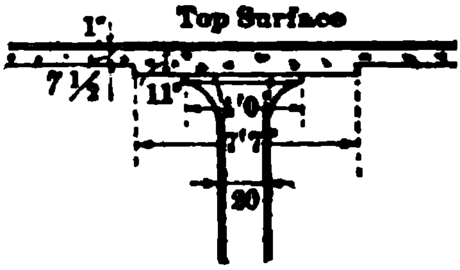


Fig. 26. Capital-head and S in Girderless Floor

bands will have 43% of 5 or 2.15 sq in, which will require twenty 3/8-in ro rods over the column, or ten on each side, lapped as above. The thicknes the slab at the middle of the span is found by the formula given above, sub tuting the proper values for the letters. The formula becomes

$$h = \sqrt{\frac{2 \times 357\,888}{0.27 \times 108 \times 600}} = \sqrt{\frac{715\,776}{17\,496}} = \sqrt{41} = 6.4 \text{ in}$$

The total depth is 6.4 + 1 in = 7.4 in, or about 7.5 in.
The width of the band = 9 ft = 108 in.
For the steel at the center of the span

$$x_1 = + \frac{M}{d_1 \times 16\,000} \text{ in which } d_1 = 0.9 h \text{ or } 0.9 \times 6.4 = 5.76 \text{ i}$$

$$x_1 = \frac{357\,888}{5.76 \times 16\,000} = 3.9 \text{ sq in}$$

The straight bands will have 66% of 3.9 or 2.57 sq in of steel which wi quire $\frac{2.57}{0.11}$ = twenty-three 3/8-in round bars or eight bars more for the m of the span than for the band set over the column.

In practice the rods are made the full length of the span, from colum column, plus the width of the drop, or in this example 20 ft + 7 ft 7 in = 27 f for the fifteen rods. Eight additional rods, 13 ft long or about the distance the edge of one drop to the edge of the next one, must be used with the fifte make the twenty-three required for the middle of the span. The diagonal b will have in the center 43% of 3.9 sq in = 1.68 sq in which require fifteen round rods or five more than one set of rods over the column. These five, ever, are to be added at the middle of the span between the drops. The are bent up over the column-head so as to be near the top of the slab to tak of the negative bending moment, the bars extending horizontally near th of the slab the full width of the drop. It is necessary to provide bent radia extending down into the column and outwards as far as the outer ring wit

as reinforcements of the column-head. The size and number of
rith the span and load; but for the floor under consideration there
ht 1-in radial rods as near the top of the slab as practicable, the
ne outer one being equal to the width of the band and that of the
g equal to the capital-head. It will be noticed in the above analysis
ıy calculations could be made certain assumptions were necessary,
ickness of the slab, which was assumed as 8 in, in order to obtain
whereas in the finished design the thickness of the slab is 7.5 in and
, which, however, does not affect the practical results materially.
ason that the design of flat slabs should be intrusted only to those
experience in the design of reinforced concrete, as good judgment
making up of a successful design; and one who is inexperienced
a specialist in this particular system of construction, if a design
to execution.

est methods of determining GIRDERLESS FLOORS is that embodied
o RULINGS GOVERNING THE DESIGN AND CONSTRUCTION OF
T SLABS, which went into effect March 1,1918. The following
ese rulings: The least dimensions of concrete columns shall be
/12 the panel-length, nor less than 1/12 the clear height of the
minimum total thickness of the slab, in inches, shall be deter-
formula, $t = \sqrt{W}/44$, in which t is the total thickness of the
and W the total live and dead load, in pounds, on the panel,
r to center of columns; but in no case shall the thickness be
(L is the panel length, center to center of columns) for floors,
/40 for roofs, nor shall a less thickness than 6 in be used. The
punching shear on the perimeter of the column-capital shall
ltimate compressive strength of the concrete. The allowable
he perimeter of the drop-panel shall be 3/100 of the ultimate
ngth of the concrete.
pose of establishing the bending moments and the resisting
quare panel, the panel shall be divided into strips known as
p B. Strip A shall include the reinforcement and slab in a
from the center line of the columns for a distance each side
equal to one-quarter of the panel-length. Strip B shall include
t and slab in the half width remaining in the center of the
angles to these strips, the panel shall be divided into similar
having the same widths and relations to the center line of the
bove strips. These strips shall be for designing purposes only,
ded as the boundary lines of any bands of steel used."
ENT COEFFICIENTS for interior panels for two-way and four-
ll panels and panels without drops or capitals, are given in
e length of panel does not exceed the breadth by more than
nputations shall be based on a square panel whose side equals
ength and breadth. In no rectangular panel shall the length
th by more than one-third of the latter. Wall columns in
ion shall be designed to resist a bending-moment of $WL/60$
$WL/30$ at the roof. Interior columns must be analyzed for
n of unbalanced loading. The Point of Inflection; Tensile
nd Compressive Stress in Concrete; Rectangular Panels,
; Rectangular Panels, Two-way System; Placing of Steel;
ler their respective headings.

CHAPTER XXVI

TYPES OF ROOF-TRUSSES

By

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1. Definitions

Use of Trusses. Whenever the distance between the side walls of a buil ing exceeds about thirty feet, and there are no intermediate walls or columns, is usually necessary to support the roof on trusses. The ceilings of large room assembly-halls, etc., also, require trusses for their support. In many cases t roof and a ceiling are carried by the same trusses.

A Truss is a framework, composed of straight, or sometimes curved, me bers or pieces so arranged that the structure as a whole acts as a beam. Sin a triangle is the only figure which cannot be changed in shape without changi the length of one or more of its sides, it follows that the pieces forming a tr must be arranged so as to form triangles. The members of a truss are usua subjected to longitudinal stresses only, either compressive or tensile. Curv members and members which act as beams supporting loads are subjected additional bending stresses. Each member of a truss is either a TIE or a STRU

A Tie is a member which has developed in it a longitudinal tensile stress.

A Strut is a member which has developed in it a longitudinal compressi stress. When vertical, struts are sometimes called POSTS or COLUMNS.

The Top Chord of a truss is composed of the upper outside members. some forms of roof-trusses top chords are called RAFTERS (Fig. 2).

The Bottom Chord of a truss is composed of the lower outside memb (Fig. 2). In roof-trusses the bottom chord is commonly called the TIE-BEA

The Web-Members are those connecting the CHORDS (Fig. 2).

A Joint is the point of intersection of two or more members of a truss (Fig.

A Panel is the distance between two adjacent JOINTS in either the upper lower chords (Fig. 2).

Purlins. Whenever possible all roof-loads and ceiling-loads should be tra ferred to trusses at the joints. This usually requires beams spanning the sp between trusses at corresponding joints. These beams, when supporting roof, are called PURLINS (Fig. 2).

2. Types of Wooden Trusses

The Simplest Truss that can be built is that shown in Fig. 1. It consi of three members forming a triangle. As the unsupported length of a st for economical reasons, should not exceed 12 feet, such a truss is not suita for spans exceeding from 20 to 24 ft; and even for a span of 20 ft there sho be a center rod, as shown by the dotted line *R*, to support the tie-beam. utilize this truss for spans greater than 24 ft, it is necessary to brace the raft from the foot of the center rod, as shown in Fig. 2. This gives us the KING-R TRUSS, the modern type of the old-fashioned KING-POST TRUSS which is sho

which was built wholly of wood except for the iron straps at *S*

Braces. When the tie-beam supports a ceiling or attic-floor, rods
ıerted at *RR*, Figs. 2 and 4, to support the load on the tie-beam.
· the number of rods and braces, as in Figs. 4 and 5, this type of
: used for spans up
d even for greater
on account of the
gth of the center
rods it is not an
rpe when the span
When there is no
tie-beam the rods
and 5, merely sup-
eam and are often

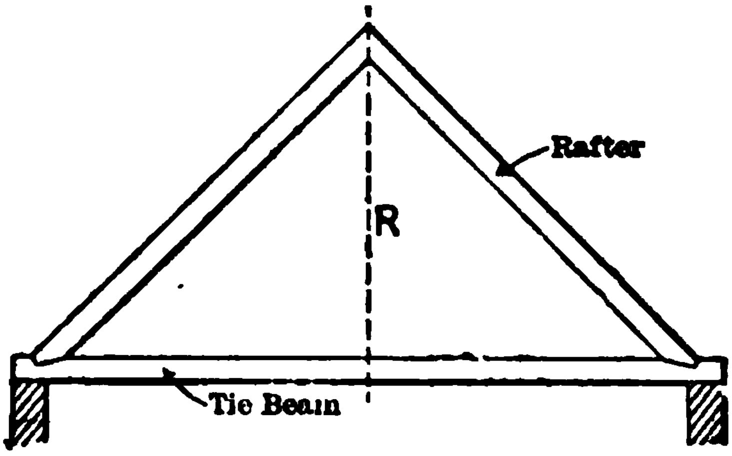
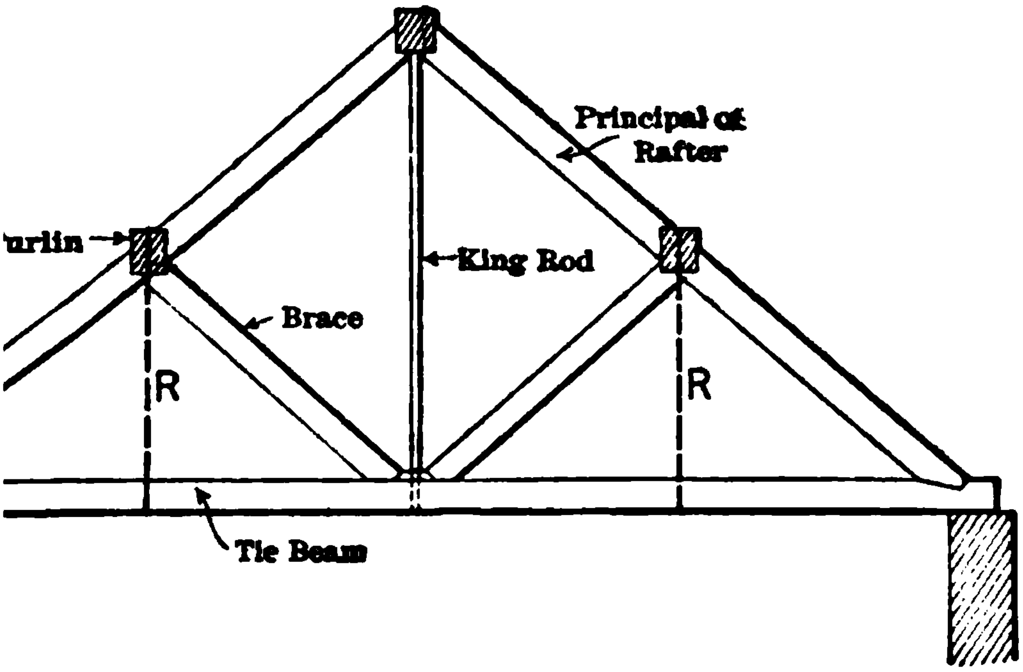


Fig. 1. Simplest Three-piece Truss. Spans up to Twenty-four Feet

Howe Trusses.
own in Figs. 4 and
imes called Howe
character of the stresses in the web-members corresponds with
esses in the web in the standard form of Howe truss. They are
ɪɴɢᴜʟᴀʀ Howe trusses to distinguish them from the standard
rith parallel chords.



ig. 2. King-rod Truss. Spans up to Thirty-six Feet

Truss. The rise of the rafter in any of the trusses, Figs. 1 to
be less than 6 in in 12 in or 26½°; a ⅓ pitch, or a rise of 8 in
ally the most economical. When the span exceeds 36 ft, it is
to cut off the top of the truss as in Fig. 6, which shows the
the ancient queen-post truss. This truss is frequently used
of deck roofs, although it may also be used for a pitched roof
When the top chord is more than 12 ft long, the size of the
considerably reduced by using a center rod and a pair of struts
7. The center rod will be especially needed if the bottom
ı is subject to a bending stress. The center rod should never
, without the braces.

Counters. The truss shown in Fig. 6 differs from those shown in Figs. to 5, in not being composed entirely of triangles and in having a rectangle the middle. Assuming the joints to be pin-connected and without friction,

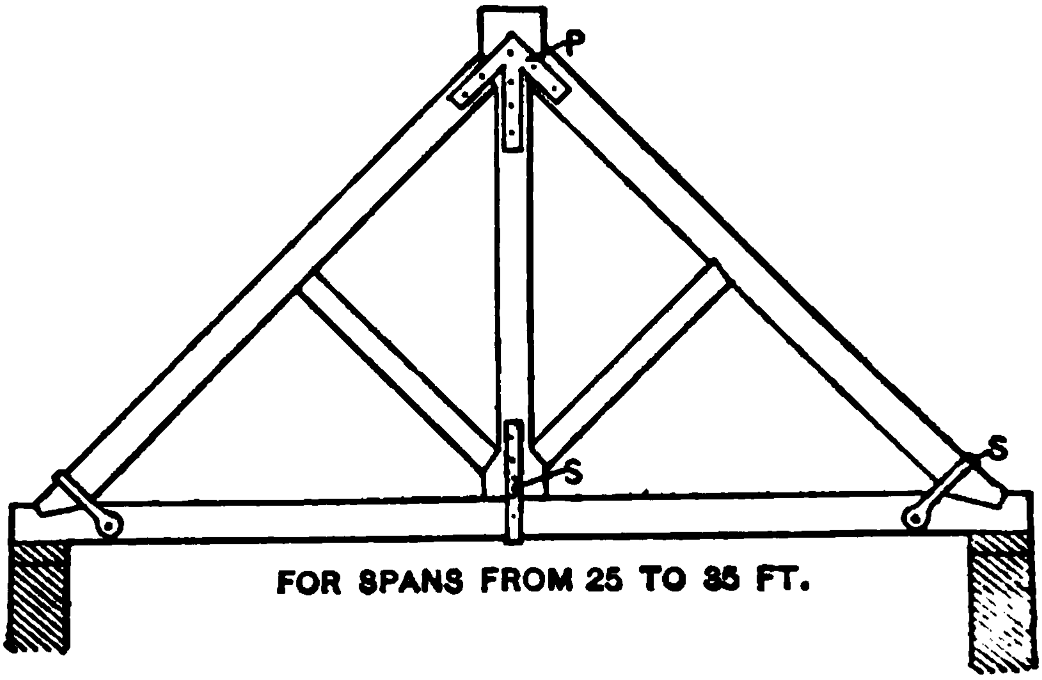


Fig. 3. Modern King-post Truss

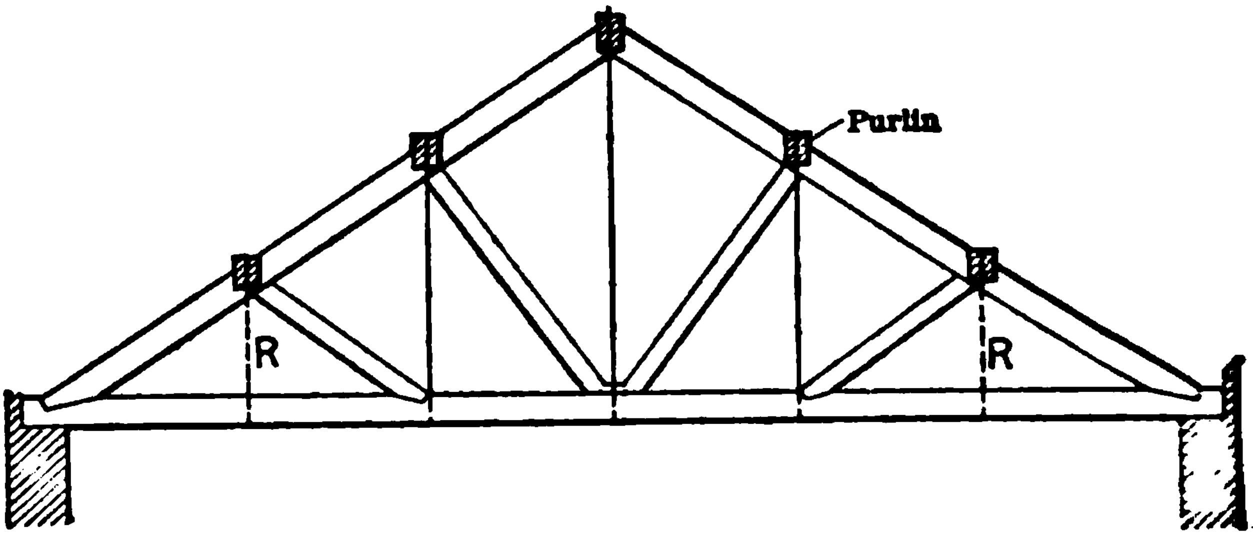


Fig. 4. Six-panel Triangular Howe Truss. Spans from Thirty-six to Fifty Feet

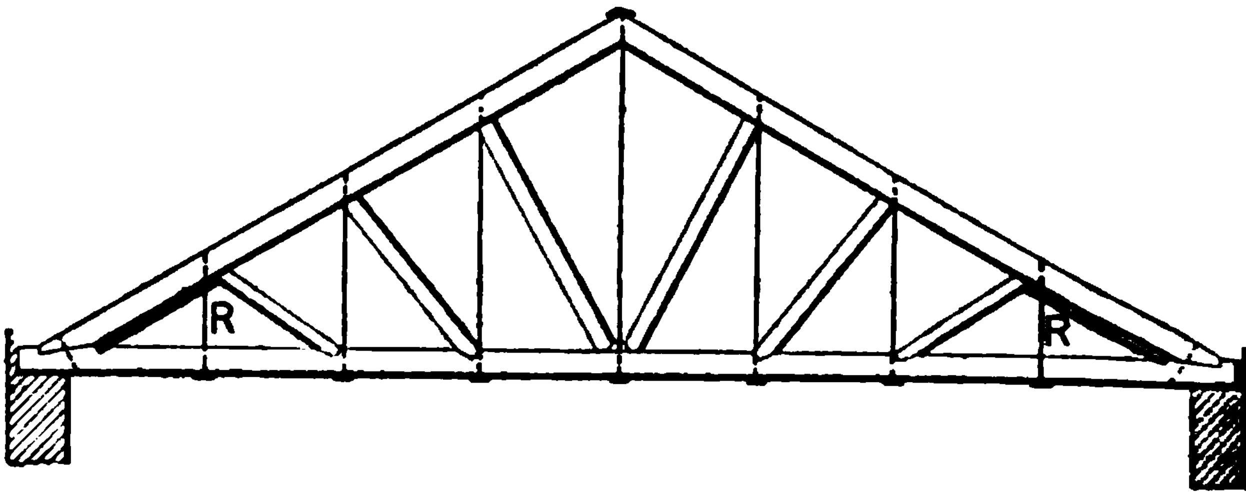
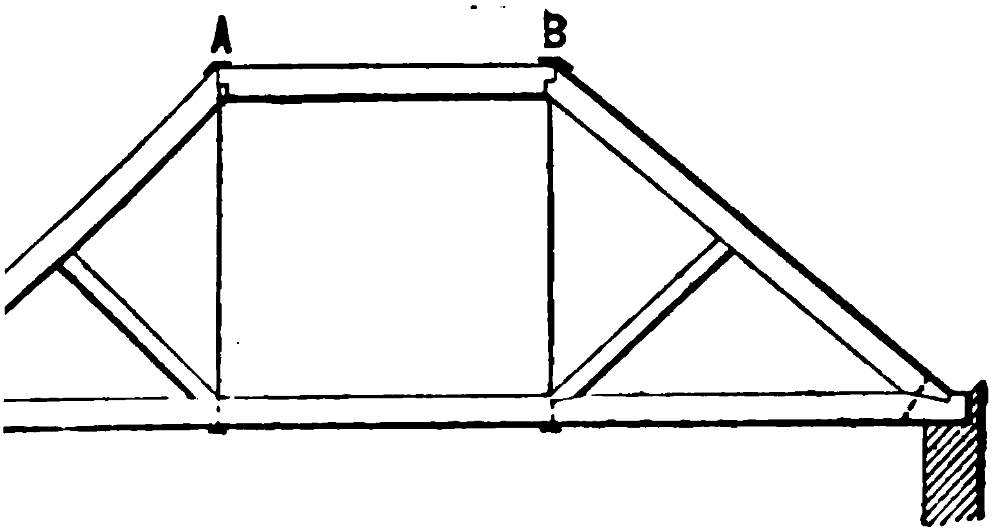


Fig. 5. Eight-panel Triangular Howe Truss. Spans from Forty-eight to Sixty Fe

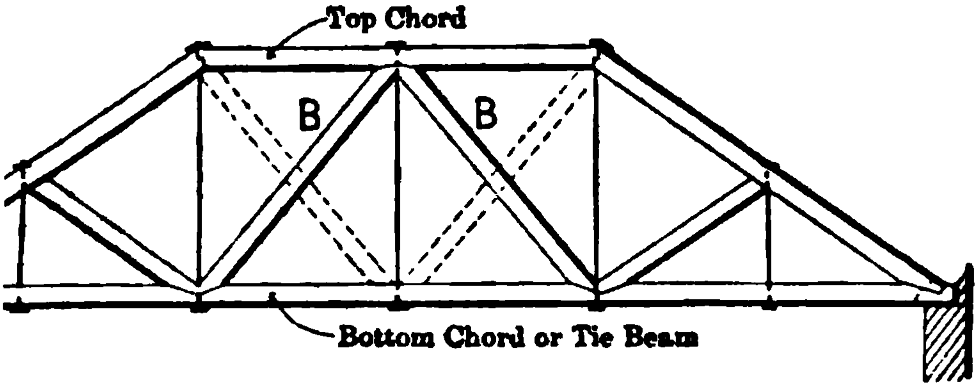
is evident that a very small inequality in the position or magnitude of the lo ing will cause the failure of the truss since the rectangle will not retain its sh This is easily verified by means of a cardboard model fastened at the jo

eyelets. When the joints at the corners of the rectangle are not :o turn they have a tendency to prevent distortion. When the



). Queen-rod Truss. Spans from Thirty to Forty-five Feet

·ly upon the left of the center the truss itself tends to assume a that shown in Fig. 8. The DISTORTION of the rectangle may be ıe introduction of a diagonal member as shown in Fig. 9. For



Queen-rod Truss. Spans from Forty to Fifty-two Feet

wn, the diagonal is in compression and is usually called a If the piece were in tension it would be called a COUNTERTIE.

al Loads. Although roof-trusses of the type shown in Fig. 9, etrical loads, do not theoretically require COUNTERS, it is never-

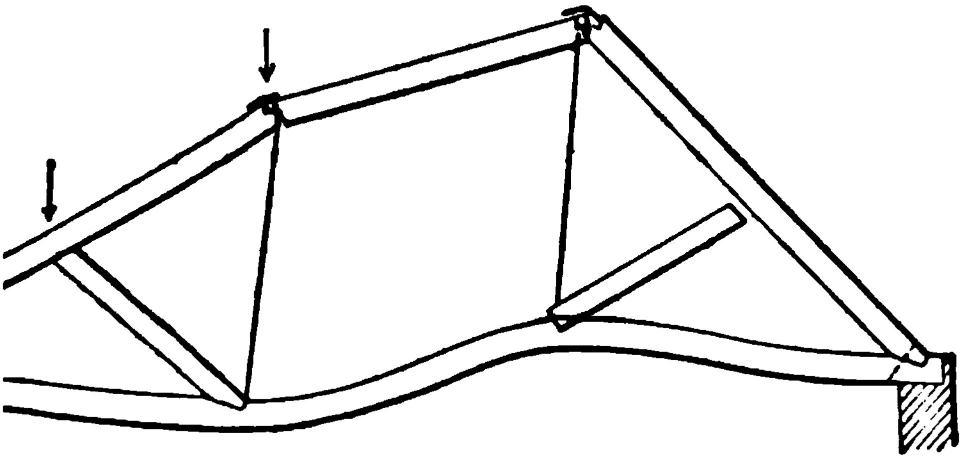


Fig. 8. Distorted Queen-rod Truss

o brace the rectangle along both diagonals to insure stability unsymmetrical loading and to relieve the joints from any latter, which is usually caused by wind, snow and floor-loads.

Reversal of Stresses. In some trusses subjected to different loading different times, the diagonal web-members near the center may be subjected tension for one loading and compression for another loading. In such case

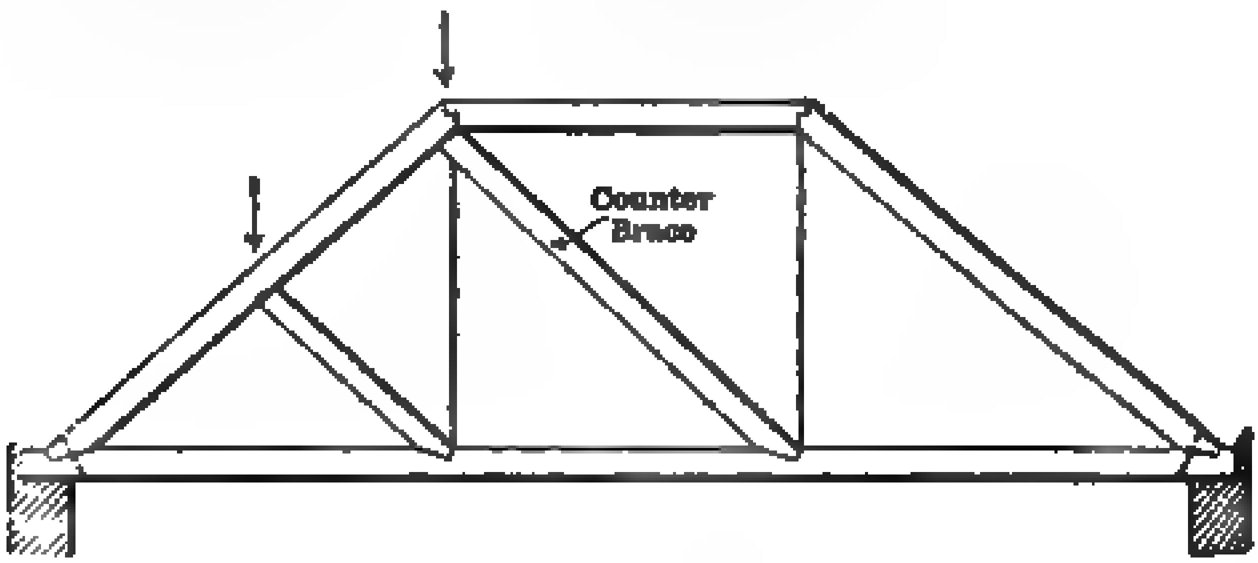


Fig. 9. Counterbraced Queen-rod Truss

is advisable to introduce a member following the other diagonal of the qua lateral containing the member subjected to the two kinds of stress, to assist main member. This piece, also, is called a COUNTERBRACE or COUNTER according to the kind of stress it has to resist. If this is not done, the mem

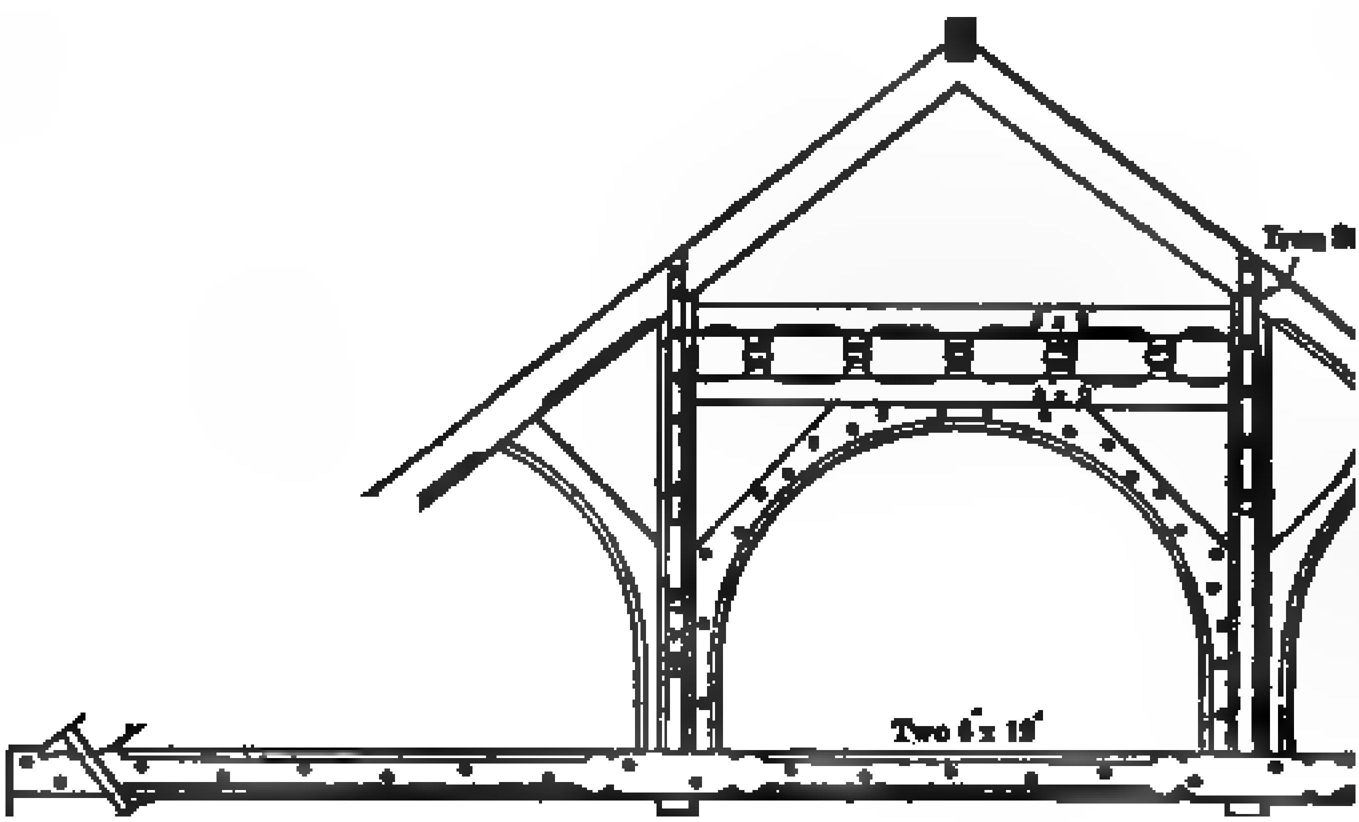


Fig. 10. Queen-post Truss. Massachusetts Charitable Mechanics' Association B ing, Boston, Mass.

which is subjected to two kinds of stress must be designed for both tension compression and the ends connected at the joints to meet the same condi

An Ornamental Queen-Post Truss, supporting a portion of the roof of Massachusetts Charitable Mechanics' Association building in Boston, Mass.,

ſr. William G. Preston, is shown in Fig. 10. The truss-members, ong-leaf yellow pine, were worked from timbers of the dimensions ıs truss wooden members instead of rods are used for the vertical ɔolted and tenoned to the tie-beam and secured to the rafters by The curved ribs take the place of counterbraces.

[1]1. Queen-rod Truss. Museum of Fine Arts, St. Louis, Mo.

Rod Truss from the Museum of Fine Arts, St. Louis, Mo., 'eabody & Stearns, is shown in Fig 11. It supports the floor s of three rods. The truss-rods have nuts and washers below the the threads on the rods are long enough to receive turnbuckles the suspension-rods with the truss. This is generally the best pending a floor from a truss.

's a detail of joint *A* of the 1.

mitted for Special Reasons. truss, sometimes used when it ep the middle part of an attic structions. In building this isable to construct the lower ers of two timbers, thoroughly , as shown. What has been to counterbraces in queen-rod also to this truss, although in continuous rafter aids very esisting distortion from wind-ıat for ordinary construction ɩot exceeding 40 ft it is safe to ıces.

Fig. 11A. Detail of Joint *A*, Fig. 11.

Supporting Common Rafters. Before describing other types ay be well to consider the manner of supporting the common russes. Occasionally it is desirable to span the common rafters russ, but as a general rule it is better construction to support s of large beams or PURLINS which themselves span from truss wn in Fig. 13.

ıe trusses can be designed so that the purlins need not be more t, and very often not more than 6 or 8 ft apart; so that the need not be more than 2 by 4 or 2 by 6 in in cross-section, while be spaced 12, 14, or 16 ft on centers. As a rule a spacing of

about 14 ft for the trusses and of 9 ft 6 in for the purlins is found to be th most economical arrangement. Another advantage in the use of purlins that where they are placed at the truss-joints no bending stresses are develop in the truss-rafters or chords; and hence the latter may be made lighter than

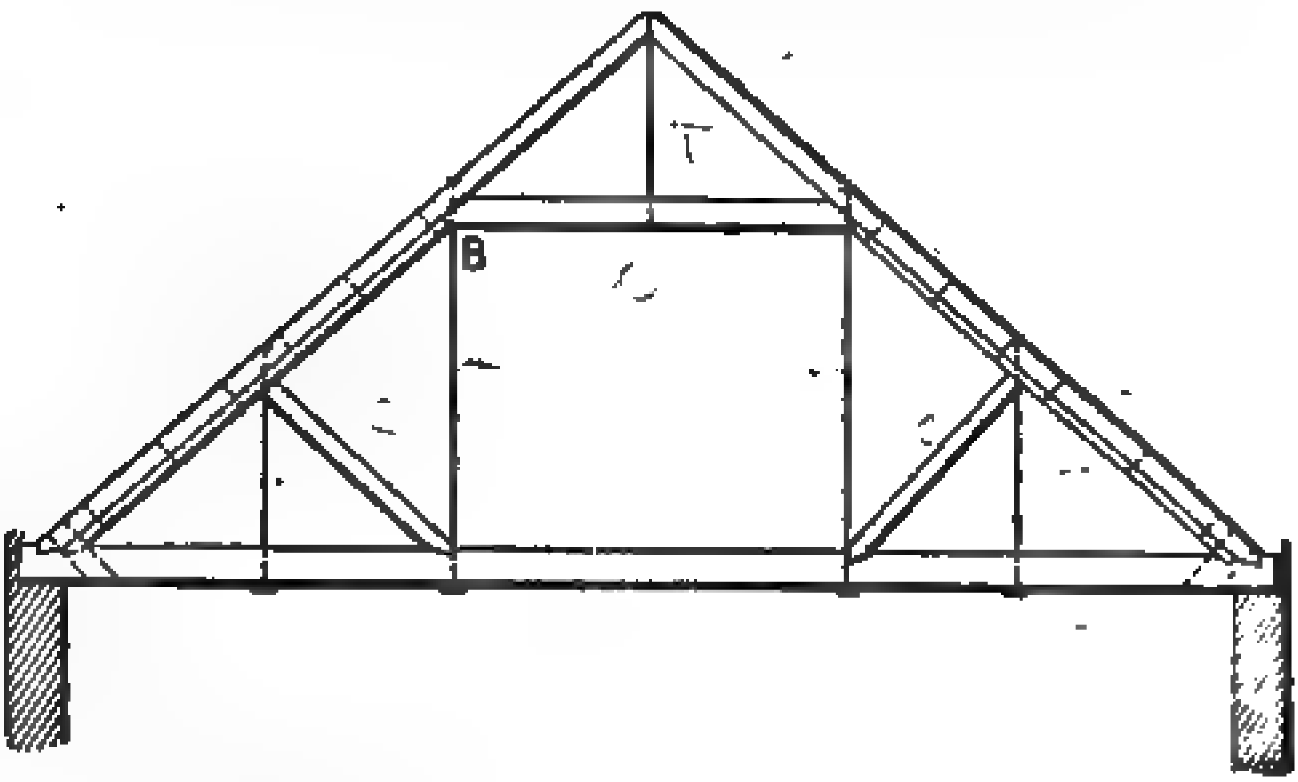


Fig. 12. Queen-rod Truss with Middle Part Clear. Spans up to Forty-two Feet

they supported the common rafters. For wooden trusses of 60 ft or great span, purlins should always be used.

Supports for Purlins. Purlins may be placed with their sides either verti or at right-angles to the plane of the roof, as shown in Figs. 2 and 13. T ends of the purlins m be supported by mea of beam-hangers, de cribed in Chapter XX by double stirrups; 3-in planks bolted a spiked to the sides the trusses, or th may rest on the t chords themselves. T ceiling-joists or flo joists are usually su ported at the sides the tie-beams, as at Fig 13, or simply r on them, as at *B*. Wh they support an at floor it is better to use the latter construction. In the case of SCISSORS TRUSS it is sometimes more economical to support the ceiling-joists by purlins; b when the tie-beams are horizontal it is more economical to use them for t direct support of the ceiling-joists or floor-joists. All chords which supp rafters, ceiling-joists or floor-joists must be designed for bending stresses well as for longitudinal stresses.

Fig. 13. Manner of Supporting Common Rafters and Purlins

Trusses with Horizontal Chords. For the support of flat roofs, with without a ceiling below, and for conditions such that horizontal trusses

types shown in Figs. 14 to 17 are undoubtedly the most satis-
den construction, when the span does not exceed 80 ft; and
ies where the cost of iron rods is relatively great, it is as econom-
this work the name HOWE TRUSS is given to this type, as it is
of the Howe bridge-truss to building-construction; and the
L TRUSS is also sometimes used. Trusses of this type can be

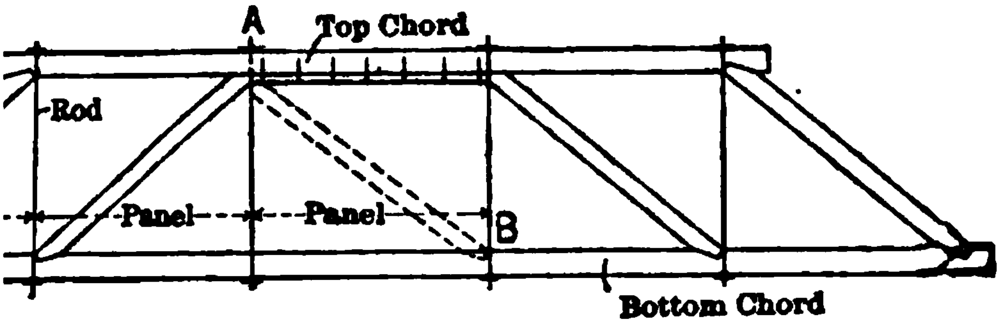


Fig. 14. Five-panel Howe Truss

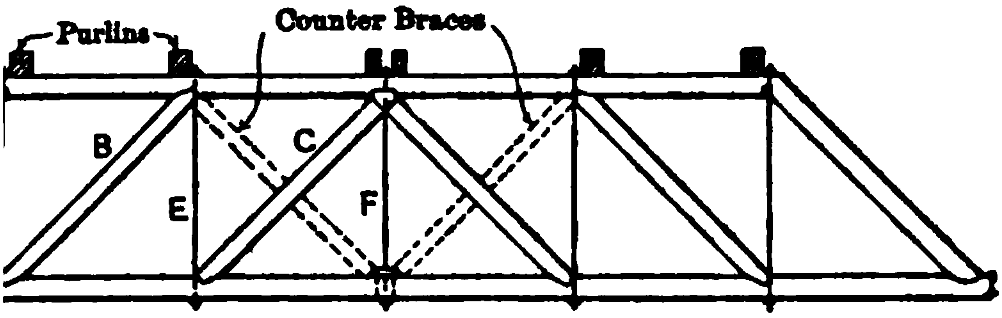


Fig. 15. Six-panel Howe Truss

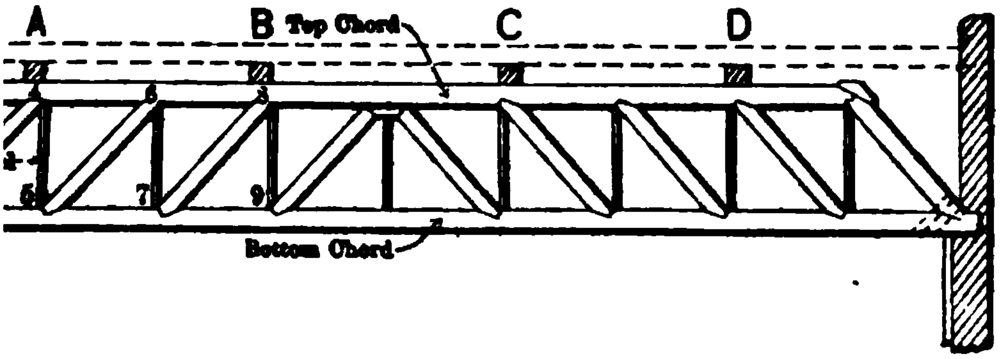
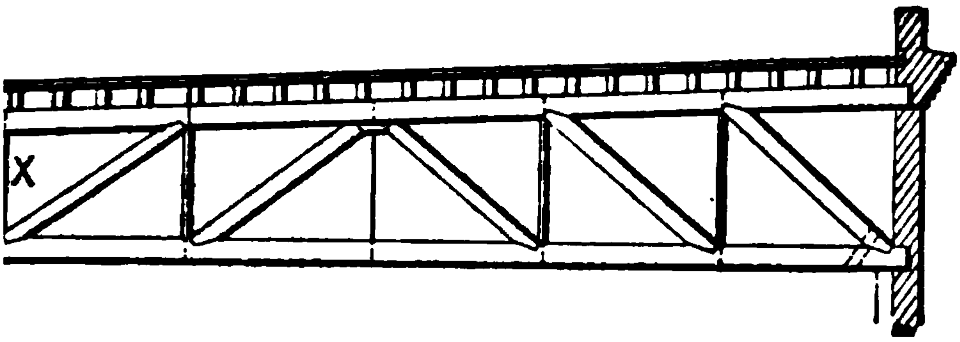


Fig. 16. Ten-panel Howe Truss



17. Six-panel Howe Truss with Top Chord Inclined

gh for spans up to 150 ft; but when the span exceeds 100 ft
per to use a steel truss of the PRATT TYPE in which the verti-
ssion and the diagonals in tension. When a Howe truss is
tudinal direction of a flat roof, the top chord may be given
he roof itself, so as to support the rafters without the blocking
7. For deck roofs the top chord may be inclined upwards
r deck-ridge, to conform to the shape of the roof, as shown in

Fig. 18. For deck roofs and mansard roofs the middle panels should ha counterbraces, as shown in Fig. 18, to resist the wind-pressure against th sides of the roof and any unequal distribution of snow.

Height of a Howe Truss. The height of the truss, measured from cent to center of the chords, should never be less than one-ninth the span f spans up to 36 ft, nor less than one-tenth the span for spans from 36 to 80 f

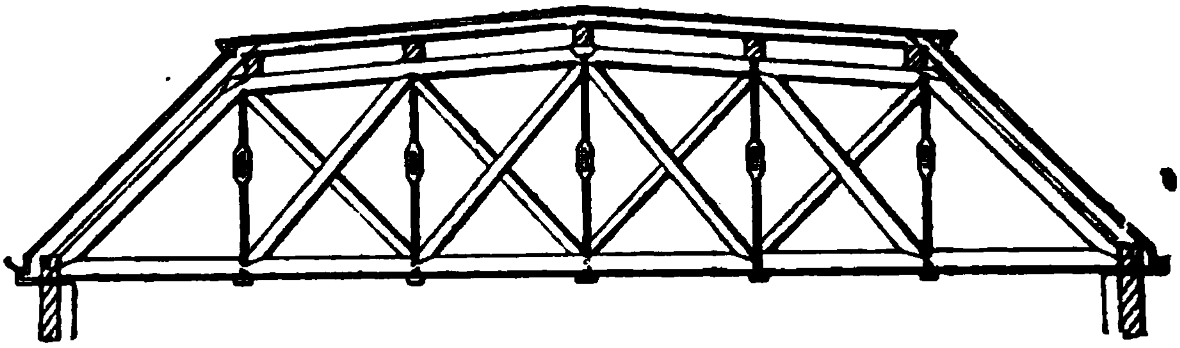


Fig. 18. Howe Truss for Deck Roofs

As a general rule a height of from one-seventh to one-sixth the span will most economical. When the top chord is inclined, as in Fig. 17, the height X, that is, at the shortest rod, should not be less than the limit given above.

Number of Panels in a Howe Truss. In this type of truss a PANEL the space between two adjacent rods or between an outer rod and the en joint (Fig. 14). As a rule, the number of panels should be such that the dia onals will have an inclination of from 36° to 60°, an inclination of about 4 being the most economical. It is not material whether there is an even or odd number of panels. If the position of one or more of the purlins is fix by some special requirement, then the panels should be so arranged that t upper joints come under the purlins, and the inclination of none of the diagon is less than 36°. Although it is generally better to have the truss symmetri about the center, it is not absolutely necessary; nor is it necessary to make t panels of uniform width. When the truss is not symmetrically loaded, howev it may be necessary to reverse the brace in one of the center panels. This po is considered in Chapter XXVII, page 1102, under the subject of UNSYMMETRI CALLY LOADED TRUSSES.

Counterbraces in a Howe Truss. If there is any chance of the tr being more heavily loaded on one side of the center than on the other, COUNTER BRACES, that is, braces inclined in the opposite direction from that of the regu braces, should be placed in the center panels, as shown by the dotted lines Fig. 15. If the truss is deep and the diagonals long it is economical to count brace each panel as shown in Fig. 18. If the number of panels is odd, as sho in Fig. 14, no diagonals are required in the middle panel when the braces and loading are symmetrical; but it is good practice to cross-brace this panel provide for any accidental unsymmetrical loading.

Spacing of Trusses. The most economical spacing, center to center, of trusses, all things considered, is usually from 12 to 16 ft for spans up to 60 and from 14 to 20 ft for greater spans.

Spacing of Purlins. Purlins should always be placed as near the tr joints as possible; they should also be spaced so as to effect the greatest econo in rafter-construction. Their spacing, therefore, determines, to a large ext the number of panels. When the height of the truss is not more than one-ni or one-tenth the span, it is often more economical to place a purlin over ev other joint, as in Fig. 16.

nensions for Six-Panel Howe Trusses, Symmetrically Loaded

r, Norway pine, Douglas fir, or eastern spruce. (See Fig. 15)

Total height		Top chord	Bottom chord	Braces			Rods not upset		
				A	B	C	D	E	F
ft	in	in	in	in	in	in	in	in	in
6	7	6× 6	6× 8	6× 6	6×4	6×3	1⅛	¾	⅝
5	2	6× 8	6× 8	6× 6	6×6	6×4			
6	8	6× 8	6× 8	6× 6	6×4	6×3	1¼	⅞	⅝
5	2	8× 8	8× 8	8× 6	6×6	6×4			
6	8	6× 8	6× 8	6× 8	6×6	6×4	1¼	⅞	⅝
5	2	8× 8	8× 8	8× 8	6×6	6×4			
7	7	8× 6	8× 8	8× 6	8×4	6×4	1¼	⅞	⅝
5	11	8× 8	8× 8	8× 6	8×5	8×4			
7	8	8× 8	8× 8	8× 6	8×5	6×4	1⅜	⅞	¾
5	11	8× 8	8× 8	8× 8	8×6	8×4			
7	8	8× 8	8× 8	8× 8	8×6	8×4	1½	1	¾
6	1	8×10	8×10	8× 8	8×6	8×4			
8	8	8× 8	8× 8	8× 8	8×6	8×4	1⅜	⅞	¾
6	8	8× 8	8× 8	8× 8	8×6	8×4			
8	8	8× 8	8× 8	8× 8	8×6	8×4	1⅜	1	¾
6	10	8×10	8×10	8× 8	8×6	8×4			
8	8	8× 8	8× 8	8× 8	8×6	8×4	1½	1	¾
6	10	8×10	8×10	8×10	8×6	8×4			
9	8	8× 8	8× 8	8× 8	8×6	8×4	1⅜	⅞	¾
7	6	8× 8	8×10	8× 8	8×6	8×4			
9	8	8× 8	8× 8	8× 8	8×6	8×4	1½	1	¾
7	7	8×10	8×10	8× 8	8×6	8×4			
9	10	8×10	8×10	8×10	8×8	8×6	1⅝	1⅛	¾
7	7	10×10	10×10	10× 8	8×8	8×4			
0	9	8× 8	8×10	8× 8	8×6	6×6	1⅜	1	¾
8	4	8×10	8×10	8×10	8×6	8×4			
0	10	8×10	8×10	8×10	8×6	6×6	1½	1⅛	¾
8	4	10×10	10×10	10× 8	10×6	8×4			
0	10	10×10	10×10	10× 8	10×6	8×6	1¾	1⅛	¾
8	4	10×10	10×10	10×10	10×6	8×6			
	6	8×10	8×10	8×10	8×6	6×6	1½	1	¾
	7	10×10	10×10	10× 8	10×6	8×6			
	6	10×10	10×10	10× 8	10×6	8×6	1¾	1⅛	¾
	9	10×12	10×12	10×10	10×8	10×6			
	6	10×10	10×10	10×10	10×6	8×6	1⅞	1¼	⅞
	9	10×12	10×12	10×12	10×8	10×6			
	2	10×10	10×10	10×10	10×6	8×6	1⅝	1⅛	¾
	10	10×10	10×10	10×10	10×6	8×6			
	2	10×10	10×10	10×10	10×8	8×6	1⅞	1¼	⅞
	0	10×12	10×12	10×10	10×8	10×6			
	4	10×12	10×12	10×12	10×8	8×6	2	1⅜	1
	1	10×12	10×14	10×12	10×8	10×6			

Bearing on Wall or Post. The point where the axial lines of the e brace and of the tie-beam intersect should always come over the support, a if possible over the axis of the supporting wall or post.

Stresses in a Howe Truss. The stresses in the chords are always greate at the middle of a truss, diminishing towards the supports, while the stresses i the rods and diagonals are greatest at the ends of a truss.

Table of Dimensions for a Howe Truss. In SYMMETRICAL TRUSS having panels of uniform width and uniformly loaded, the stresses in the diffe ent members are proportional to the span, number of panels, height of trus spacing of trusses and load per square foot. It is therefore possible to prepa tables giving the proper dimensions of the members of such trusses. Table gives the dimensions for six-panel trusses for heights of one-sixth and one-eight the span and for three different spacings. These dimensions are for a fl roof covered with tin, sheet iron, or composition; a snow-load of 16 lb per sq f equivalent to about 24 in of light, dry snow; also for a lath-and-plaster ceilin supported by the bottom chord. The chords and braces are of Norway pin and the rods of wrought iron. These dimensions apply only when the rafte

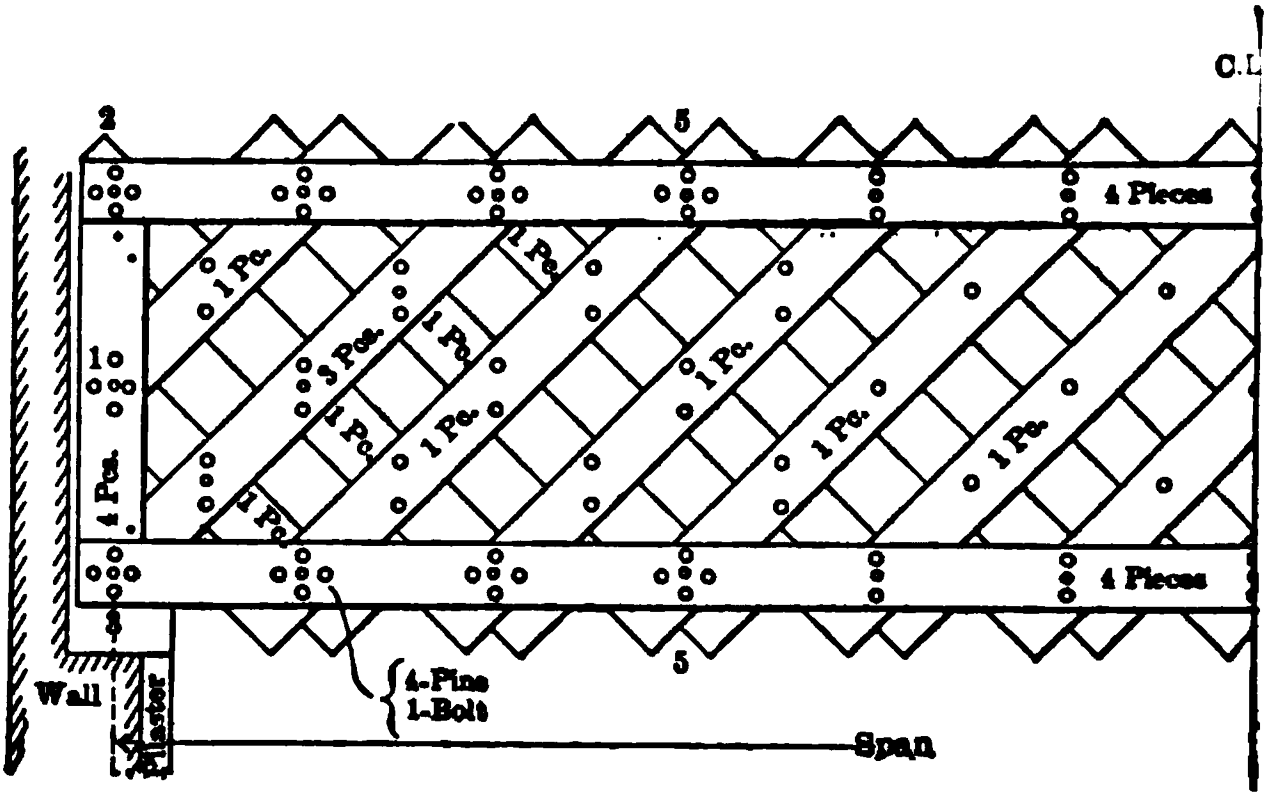


Fig. 19. Lattice Truss

are supported on purlins placed at the upper joints, as in Figs. 15 and 16 When the rafters rest on the top chord, as in Fig. 17, the dimensions o the latter must be increased and special calculations made for it. The dimen sions given in the table may be used for trusses of greater height than tho given, but not for trusses of less height, as the less the height the greater th stresses in the chords and braces. When the conditions of load, span, heigh and spacing are not exactly as given above and in the table, the stresses shoul be determined and the members of the truss proportioned accordingly; bu even in such cases the table will serve somewhat as a check on the com putations.

Lattice Trusses. In localities where timber is not expensive the LATTIC TRUSS (Fig. 19) is often found economical for supporting flat roofs. This typ of truss was invented for bridges by Ithiel Towne in 1820 and a large number o

; have been constructed with trusses of this type, some of which now (1915) in New England. The principal objections to the ıdency to twist sidewise, like a thin board on edge, its flexibility lane and the difficulty of getting sufficient bearing material at As indicated in Fig. 19, the truss is composed of top and bottom parallel, connected by lattice bracing. The chords are com-lanks, two being on one side and two on the opposite side of the bottom chord the planks should be as long as can be obtained o that no two splices are near the same point. The available om chord to resist tension is the area of three planks less the area oints by the connecting pins or bolts. Each member of the web gle plank arranged as shown in Fig. 19. The braces are inclined ıbout 45° and usually three sets are sufficient, as shown in the nnections are best made with American-locust pins, which give eas without much extra weight. Modern construction employs : expensive and add considerable weight. There should be at

al Section of
in Fig. 19

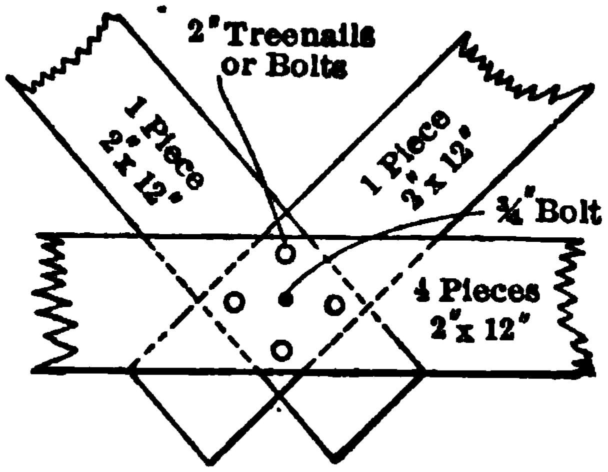


Fig. 21. Lower Joint 5 of Truss Shown in Fig. 19

each connection, if the planks are wide enough to permit, and t the chord-joints. Since about one-half the web planks resist the web projects beyond the chord at least 4 in to provide dinal shearing area. The ends are reinforced by vertical tim-:en the chords and each set of diagonals is thoroughly fastened

In some cases it is necessary to add timbers on the outside d them down to the lower face of the bottom chords to relieve bearing-stresses where they rest on the supports. The methods he stresses in this truss are considered in Chapter XXVII, 91. Figs. 20 and 21 show details of this lattice truss.

ses with Raised Bottom Chords. All of the trusses thus e horizontal bottom chords; and this construction is the most as the most economical and should be used whenever condi-ssitate a greater height of ceiling. In roofing churches, public ceilings are often desirable as they increase the general height t increasing the height of its side walls.

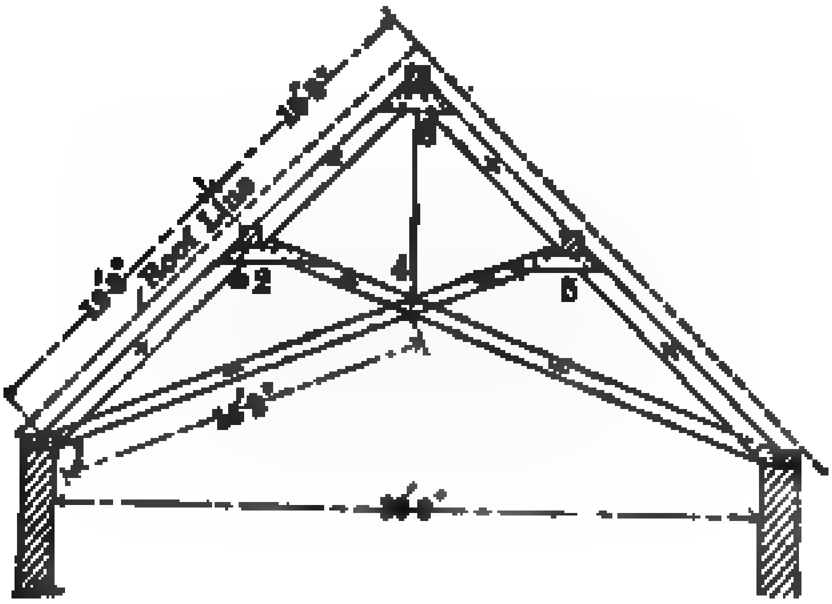
Table II. Dimensions for Lattice Trusses, Uniformly Loaded

Timber, Norway pine, Douglas fir, and yellow pine. (Fig. 19)

Span	Spacing of trusses	Height out to out of chords	No. of spaces	No. and size of pcs of bottom chord	No. and size of pcs of top chord	Size of braces	No. and diameter of treenails or bolts, joints 1–5, Fig. 2
ft	ft	ft in		in in	in in	in in	in
40	12	5 6	16	4 2×6	4 2×6	2×6	4 1
		7 2	12	4 2×6	4 2×6	2×6	4 1
	14	5 7	16	4 2×6	4 2×8	2×6	4 1
		7 3	12	4 2×6	4 2×8	2×6	4 1
	16	5 8	16	4 2×8	4 2×8	2×8	4 1¼
		7 4	12	4 2×8	4 2×8	2×8	4 1¼
50	12	6 8	16	4 2×8	4 2×8	2×10	4 1¼
		8 8	12	4 2×8	4 2×8	2×10	4 1¼
	14	6 8	16	4 2×8	4 2×8	2×10	4 1¼
		8 8	12	4 2×8	4 2×8	2×10	4 1¼
	16	6 9	16	4 2×8	4 2×10	2×10	4 1¼
		8 8	12	4 2×8	4 2×8	2×10	4 1¼
60	12	8 4	16	4 2×10	4 2×10	2×10	4 1¾
		10 10	12	4 2×10	4 2×10	2×10	4 1¾
	14	8 4	16	4 2×10	4 2×10	2×10	4 1¾
		10 10	12	4 2×10	4 2×10	2×10	4 1¾
	16	8 4	16	4 2×10	4 2×10	2×10	4 1¾
		10 10	12	4 2×10	4 2×10	2×10	4 1¾
70	14	9 5	16	4 2×10	4 2×12	2×10	4 1¾
		12 4	12	4 2×10	4 2×10	2×10	4 1¾
	16	9 5	16	4 2×10	4 2×12	2×10	4 1¾
		12 4	12	4 2×10	4 2×10	2×10	4 1¾
	18	9 6	16	4 2×12	4 2×12	2×10	4 2
		12 6	12	4 2×12	4 2×12	2×10	4 2
80	14	11 0	16	4 2×12	4 2×12	2×12	4 2
		14 0	12	4 2×12	4 2×12	2×12	4 2
	16	11 2	16	4 2×14	4 2×14	2×12	4 2
		14 0	12	4 2×12	4 2×12	2×12	4 2
	18	11 2	16	4 2×14	4 2×14	2×12	4 2
		14 1	12	4 2×12	4 2×14	2×12	4 2

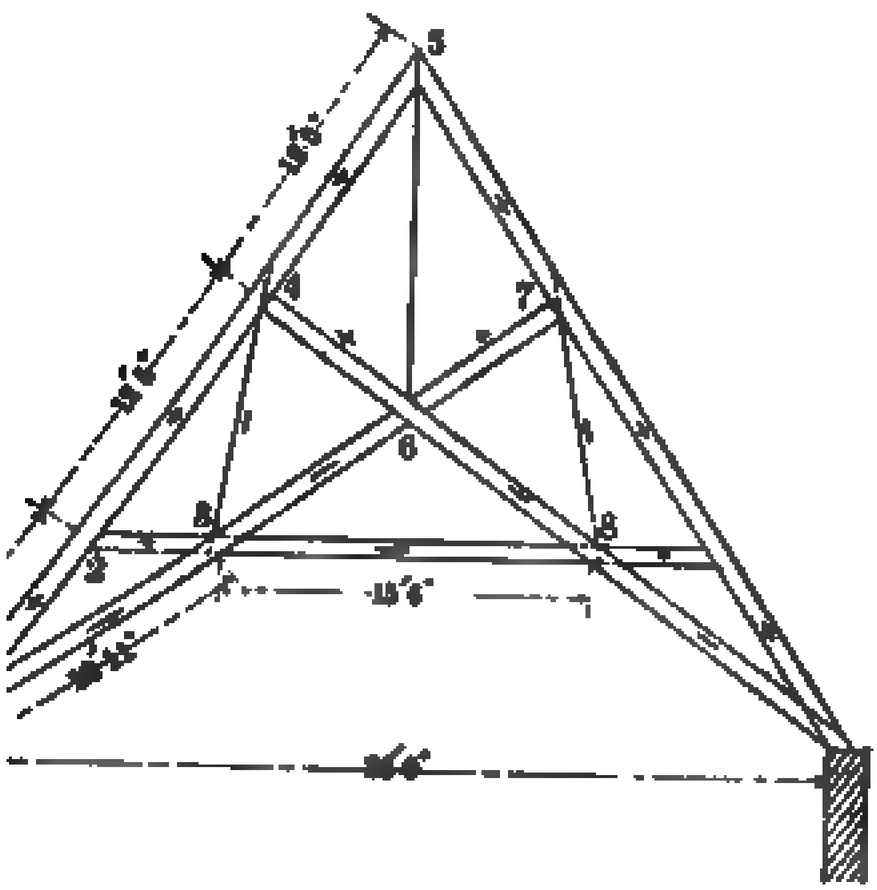
Note. All joints should be thoroughly spiked and packing blocks used where necessary. When treenails are used each chord-joint should have in addition one ¾-in bolt as shown in Fig. 21.

Scissors Trusses. For the roofs described in the preceding paragraph some form of the SCISSORS TRUSS, so named from its resemblance to a pair of scissors, is most often used. When correctly designed, with members of the proper size, and with joints carefully proportioned to the stresses, it is a very good truss for supporting roofs over halls and churches, up to a span of 48 ft; but for greater spans it should be used with caution, as the stresses become very great and the joints difficult to make. Figs. 22 to 27 show different forms of this truss and modifications of it adapted to different spans and roof-pitches. None of these trusses exerts a large horizontal thrust if the members are of ample size and the joints properly made. The members having a plus sign e



g. 22. Simple Scissors Truss. Spans up to Thirty Feet

. 23. Scissors Truss. Spans Exceeding Thirty Feet



Truss. For Steep Roofs. (See Chapter XXVIII, Figs. 18 and 19)

or close to them are in COMPRESSION, while those having a minus sign are TENSION. The determination of the actual HORIZONTAL THRUST is consider on pages 1085–1087. The members indicated by a single line should be rod

Fig. 25. Modified Scissors Truss. For Medium Pitch. (See, also, Chapter XXVI Figs. 18 and 19)

except in the case of bottom chords. Fig. 22 shows the simplest form of t SCISSORS TRUSS, which is suitable for spans up to 30 ft. When the span excee 30 ft, it is more economical to use two purlins on each side to support the co

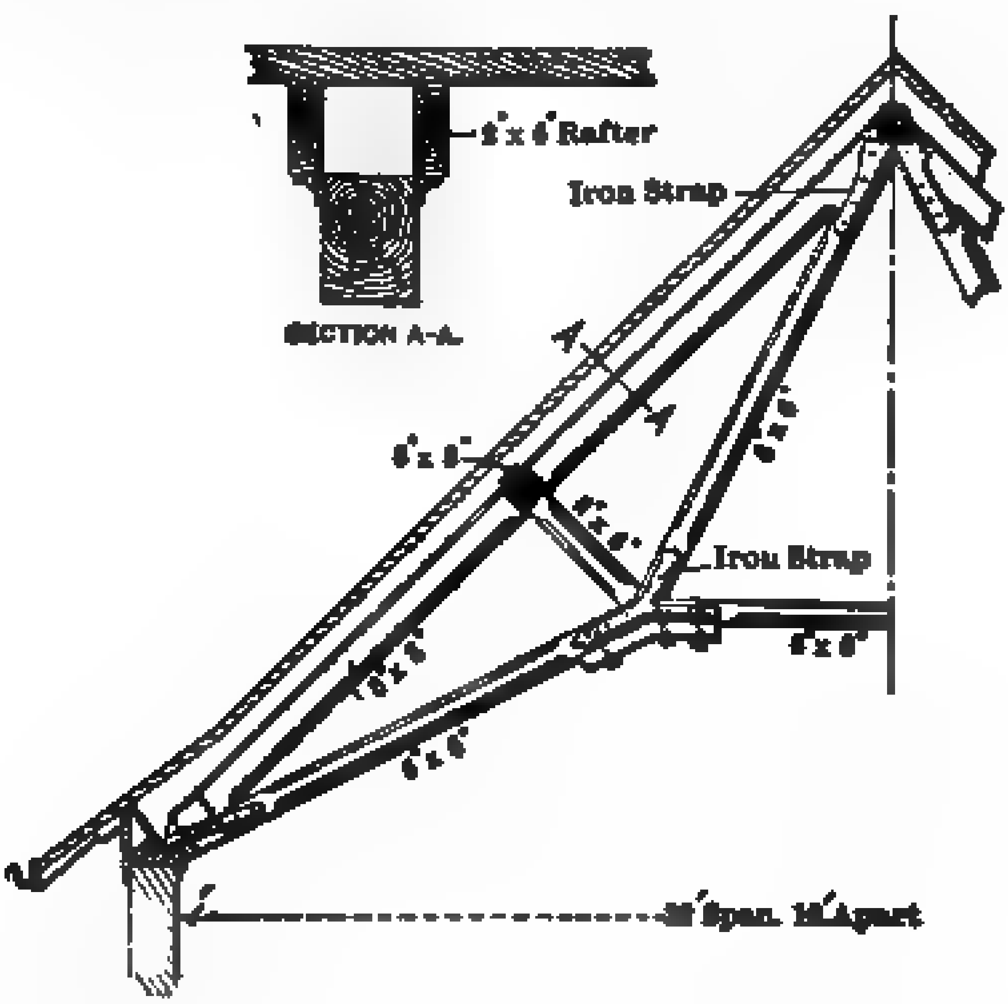
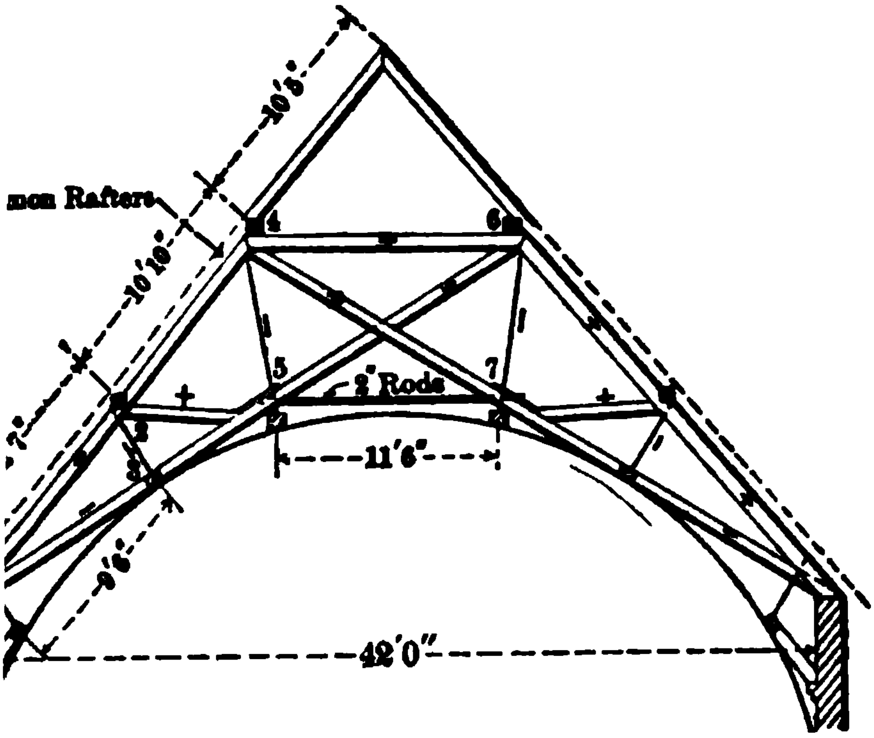


Fig. 26. Finished Cambered Truss. (See, also, Chapter XXVIII, Figs. 18 and 19)

mon rafters; and additional supports from the bottom chords are general required, calling for additional rods and braces, as shown in Fig. 23. For steep roof the arrangement shown in Fig 24 is generally the best, and for

t shown in Fig 25, in which the scissors pieces do not cross nor Fig. 26 shows a finished truss, built on somewhat the same lines wn in Fig. 25 but with only one purlin. This truss can hardly a scissors truss but is shown here for convenience. It is really as that of the truss shown in Fig. 33. . The truss shown in Fig.) that shown in Fig. 24, with the peak cut off, but for spans



Scissors Truss. Spans Exceeding Thirty-six Feet. (See, also, Chapter XXVIII, Figs. 18, 19 and 20)

is more economical. It can also be used where the roof is is form it is better to use ceiling-purlins to support the ceiling-n the latter from truss to truss.

m Trusses. Two of the principal characteristics of the Gothic ure are the relatively elaborate ornamentation of structural posure to view of the construction of a building as a whole. rch and steep roof were developed the roof-truss became an in the ornamentation as well as in the construction of Gothic s. The trusses of this period were built almost entirely of lly of very heavy timbers, to give the appearance of great the most common types of these Gothic trusses, and also ital, was the HAMMER-BEAM truss, still often used in churches othic style. Figs. 28 and 29 show early English forms of this s its name from the horizontal beam H, called the HAMMER-of the principal rafter. In the more ornamental trusses this s usually carved to represent royal personages or angels. er in principle from those thus far described, in having no no substitute for one. In fact the trusses shown in Figs. come within the scope of the definition of a TRUSS given at the chapter. Although the rafters or principals are connected truss by a short COLLAR-BEAM, this offers but little resistance f the rafters to spread at their lower ends; and hence the er upon the transverse strength of the rafters or upon the alls to keep it intact and, generally, upon both. This form of t of an ARCH, as vertical loads produce inclined reactions at the alls and churches of the Gothic period the walls were generally

very thick and usually reinforced on the outside by BUTTRESSES built agai them and directly opposite the roof-trusses. In most cases such a wall posses sufficient stability to withstand the THRUST of the truss, and hence the bott chord may be dispensed with; but in a wooden building the walls, unless t at the top, offer no resistance whatever to being thrust out and hence, in s buildings, no truss which exerts an outward thrust on the walls should be u

Fig. 28. Hammer-beam Truss. Early English Form

It is therefore generally impracticable to use a hammer-beam truss in a woo building. Where these trusses are used, the CEILING is generally formed of matc sheathing, nailed to the under side of the JACK-RAFTERS between the pur thus allowing the latter to be seen. The purlins are generally decorated, FALSE RIBS are often placed vertically between them, to divide the ceiling PANELS. The main rafters should be made very large to prevent them breaking at the point A, Figs. 28 and 29.

First Church, Boston, Mass. An excellent example of a truss adapted to modern conditions is shown in Fig. 30, which half of one of the trusses designed by Ware & Van Brunt, for

29. Hammer-beam Truss. Early English Form

oston, Mass. The truss is finished in black walnut and has very strong and heavy. Fig. 31 shows the framing of the : the casing and FALSEWORK It should be noticed that lumn in the upper part of the truss, Fig. 30, there is an iron

Fig. 30. Hammer-beam Truss. First Church, Boston, Mass.

hich resists the tensile stress. In this form of truss the line of t of the arch enters the wall just above the CORBEL, *K*; and, as inclined only about 30° from the vertical, its tendency to over- is not very great, and may be resisted, in this particular case,

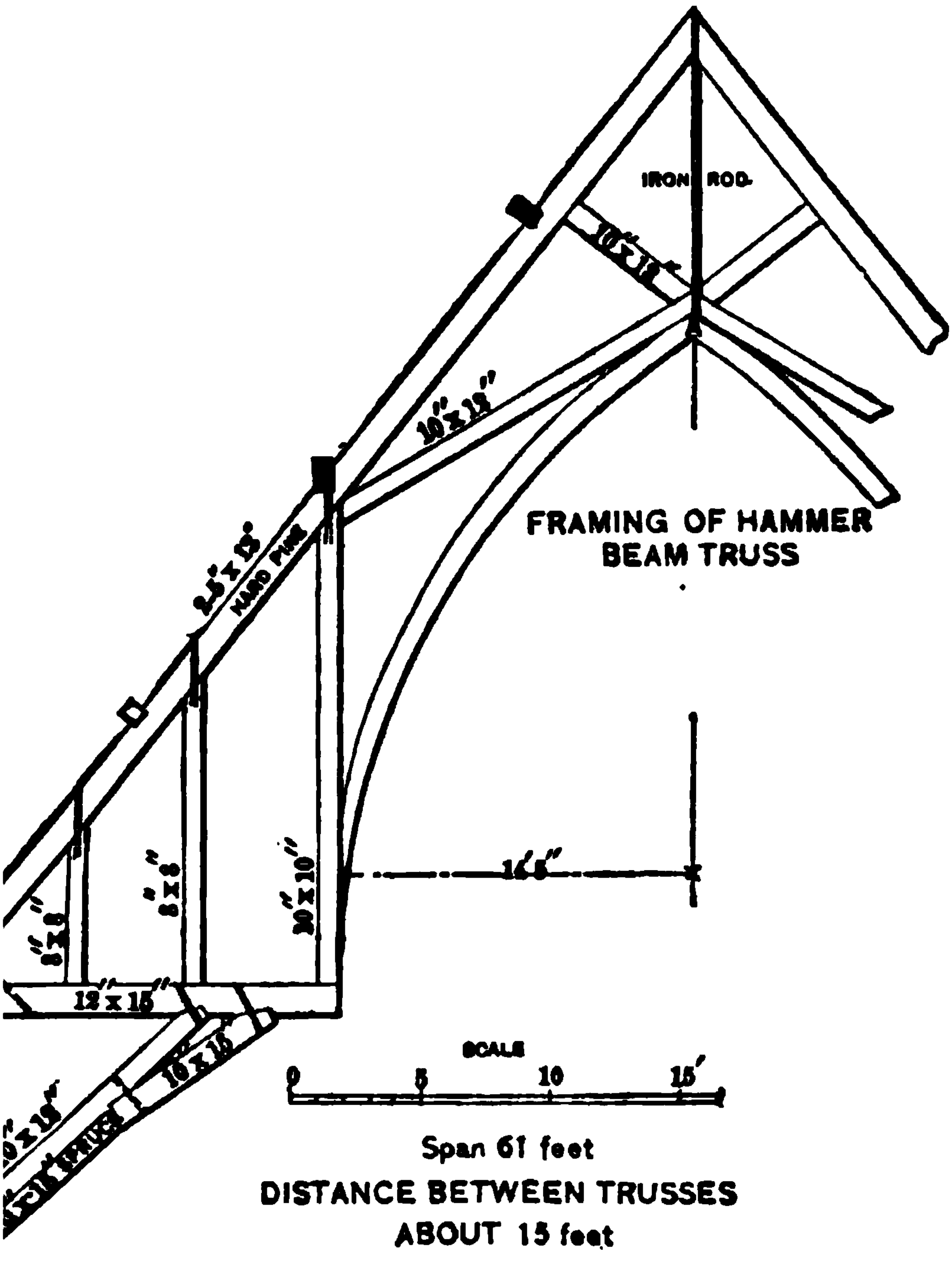


Fig. 31. Framing of Truss Shown in Fig. 30

2 ft thick, thoroughly reinforced by a BUTTRESS of proper the outside. In trusses of this kind, the various members fastened together wherever they cross or touch each other, a whole made as rigid as possible. No dependence should casings and panel-work for any extra strength.

Truss for Emmanuel Church, Shelburne Falls, Mass. Fig. 32 sho
another form of truss designed by Van Brunt & Howe, for Emmanuel Chur
Shelburne Falls, Mass. It is probably A VARIATION OF THE HAMMER-BEA

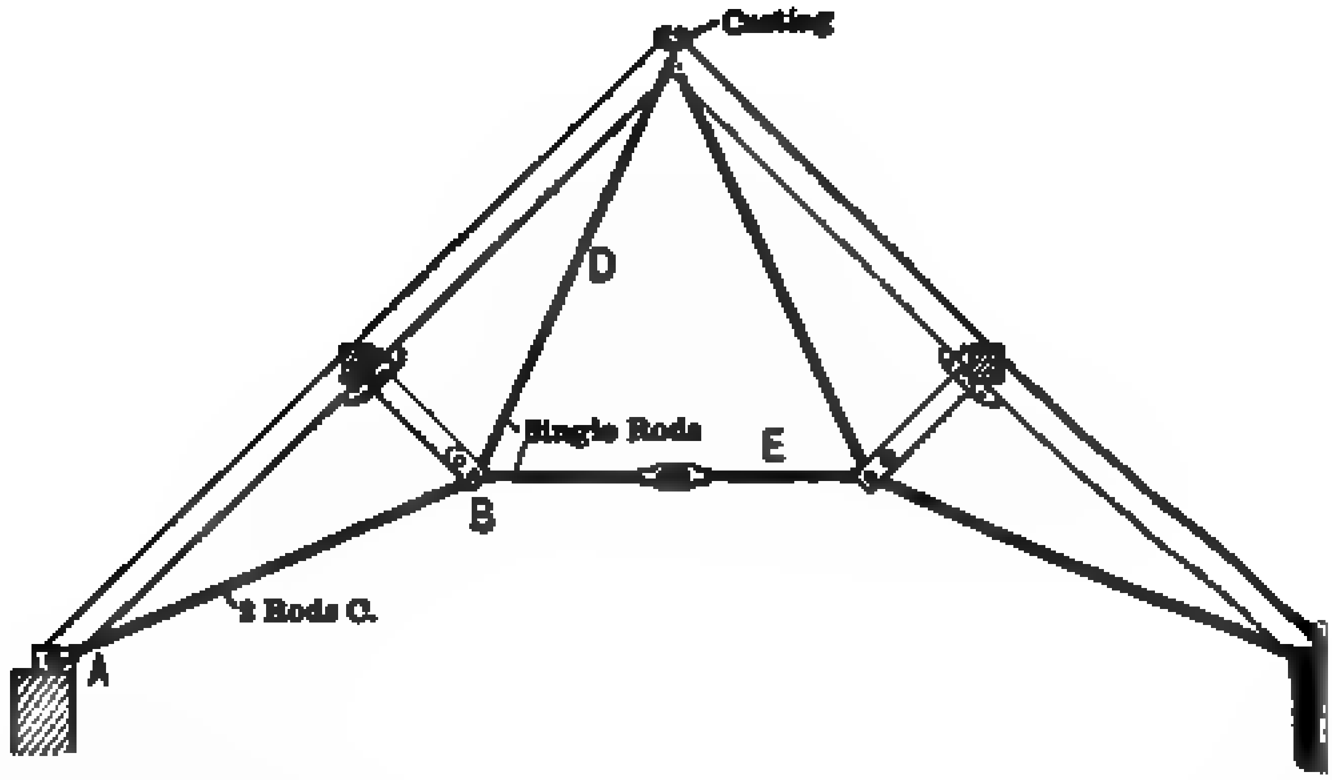
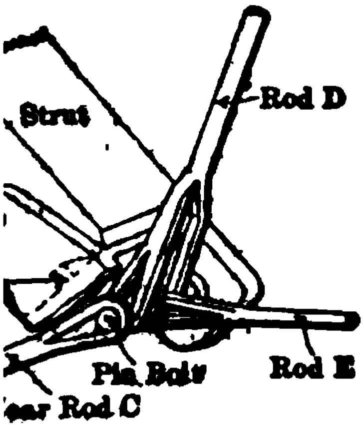


Fig. 33. Wooden Truss with Iron Ties. Spans up to Thirty-six Feet

form and when securely bolted together at all the joints can be design
to exert very little thrust on the walls. The rafters and cross-tie are
formed of two pieces of timber, separated but bolted together, the
upright members passing between these pieces. The hammer-beams are

ngels. The action of the stresses in hammer-beam trusses is
iapter XXVII, pages 1087 to 1089.

usses with Iron Ties. Where there is no ceiling beneath the
lesirable to make the trusses as light in appearance as possible,



Detail of Joint B, Fig. 28

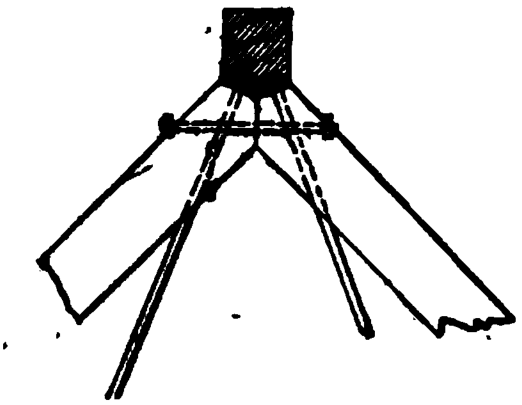


Fig. 33a. Alternate Detail of Joint at Ridge, Fig. 33

steel rods may be used for the ties, and the wooden rafter-
ts retained. For moderate spans such trusses are cheaper than
nd where the rafters and purlins are of wood they are about as
and 34 show forms of trusses well adapted to many roofs. The
n in Fig. 34 are for yellow-pine or Douglas-fir timber and wrought-

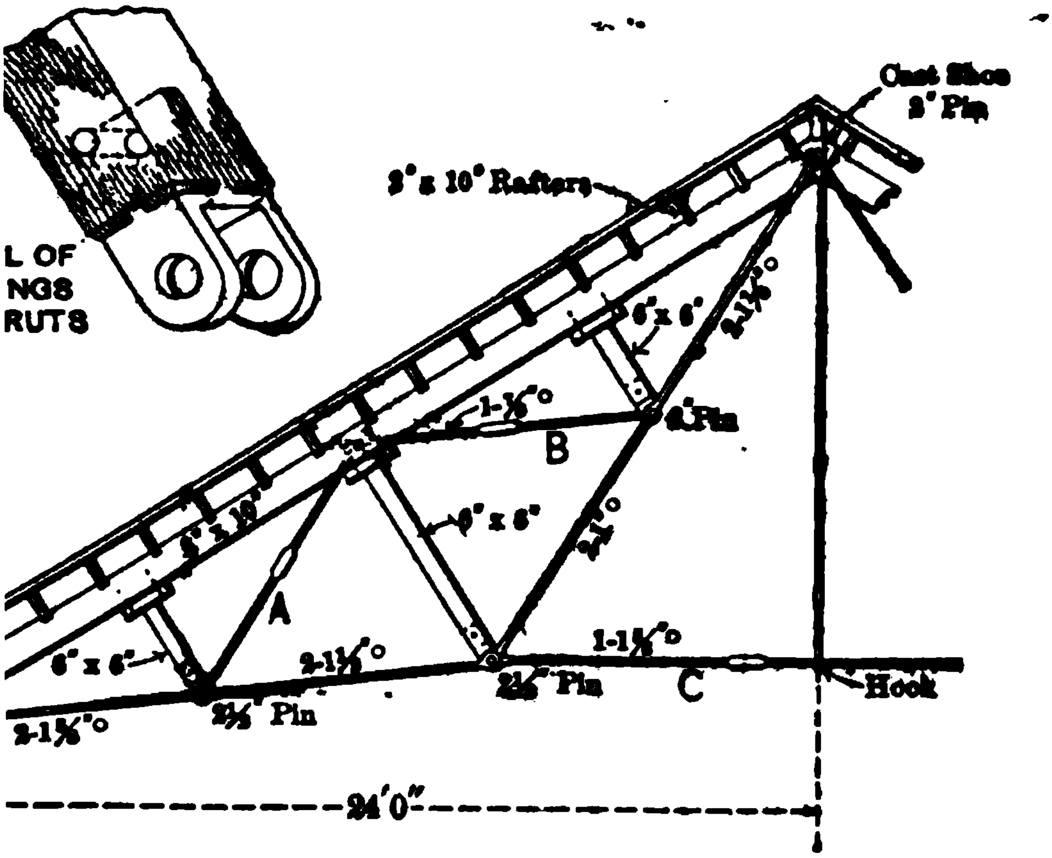


Fig. 34. Wooden Truss with Iron Ties

re ample for a slate roof, the trusses being spaced from 12 to
. Trusses of the form shown in Fig. 33 are sometimes made
C and D continuous. They should not be made in this way,
the entire rod is proportioned for the stress in C, as this stress
that in D. The best construction for the joint B is illustrated

in Fig. 33A, which shows a CAST-IRON SHOE fitted to the end of the strut t receive the pin. For the truss shown in Fig. 34, a shoe made as shown in th detail drawing makes a better connection for the rods, two of the latter bein placed outside of the brackets and three between them. For a truss with single strut, a TURNBUCKLE on the rod E serves to tighten the rods. Whe there are three struts, the should be five turnbuckles, in Fig. 34. A cast-iron sho should be made to receive th foot of the rafter and the ro secured to a pin passed throug shoe and rafter. At the ape also, of the truss shown in Fi 34, there should be castings t receive the ends of the rafter and pins for the tie-bars. Th apex-joint of the truss (Fi 33) may be made either b crossing the rods through CAST WASHER, or as shown i Fig. 33B. The pins at th joints should be computed fo shear, bearing and flexur More modern construction r places the cast iron shown with STEEL PLATES and PINS. When a hamme beam truss is to be supported on a clerestory-wall which is not very thi nor braced from the outside, a truss of the form shown in Fig. 35 may b used to advantage. It has the appearance of a hammer-beam truss and wh placed over a high nave the effect of the rods is not objectionable. The tie-rods should extend through the hammer-beams to their outer ends.

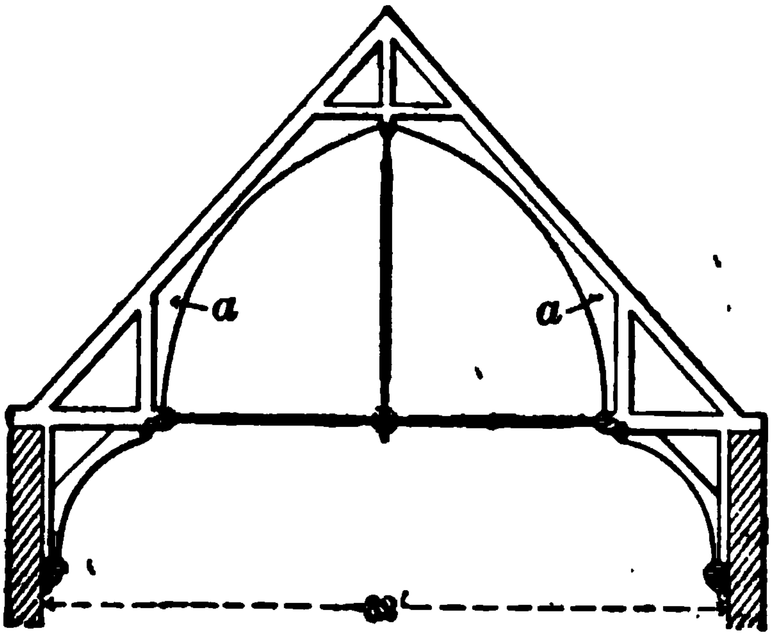


Fig. 35. Hammer-beam Truss for Grace Chapel, New York City

Truss for Grace Chapel, New York City. The CURVED RIBS *a*, *a*, Fig. 35, have a tendency to bend at their smallest section and BRACES under the hammer-beams are necessary to prevent vertical deflection in the latter. A truss similar to this was used in Grace Chapel, New York City.

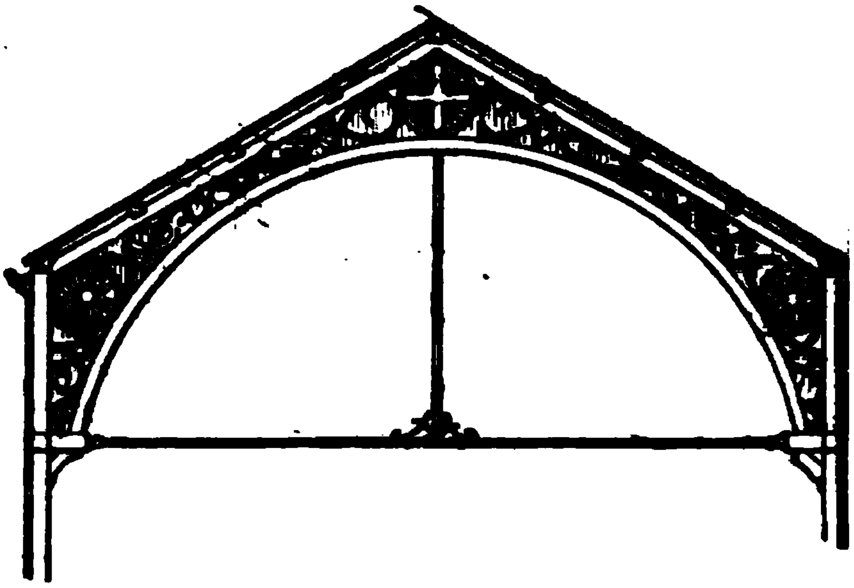


Fig. 36. Truss for Metropolitan Concert Hall, New Yo City

Truss for Metropolitan Concert-Hall, New York City. Fig. 36 sho a form of truss used to support the roof of the Metropolitan Concert-Hall, Ne York City, George B. Post, architect. The span is about 54 ft and the propo tions are about as shown. The arch between rafters and raised rib is orn mented with sawed work and the truss has a very light and airy appearan The tie-rod is kept from sagging by a vertical rod from the crown of th arch.

rched Ribs with Iron or Steel Ties. For roofing large halls SEGMENTAL TIMBER ARCH, with an iron or steel tie to take up the ust, is about the cheapest construction, especially where there is e supported. Figs. 37 and 38 are good examples of this form of

Fig. 37. Segmental Timber Arch

ED RIBS supporting all the load and the TIE-RODS preventing the from the spreading which would result without them.

. C. M. A. Building, Boston, Mass. This truss is shown in framework shown above the arch is simply to support the purand carry the load directly to the arch. It does not assist the in carrying the load

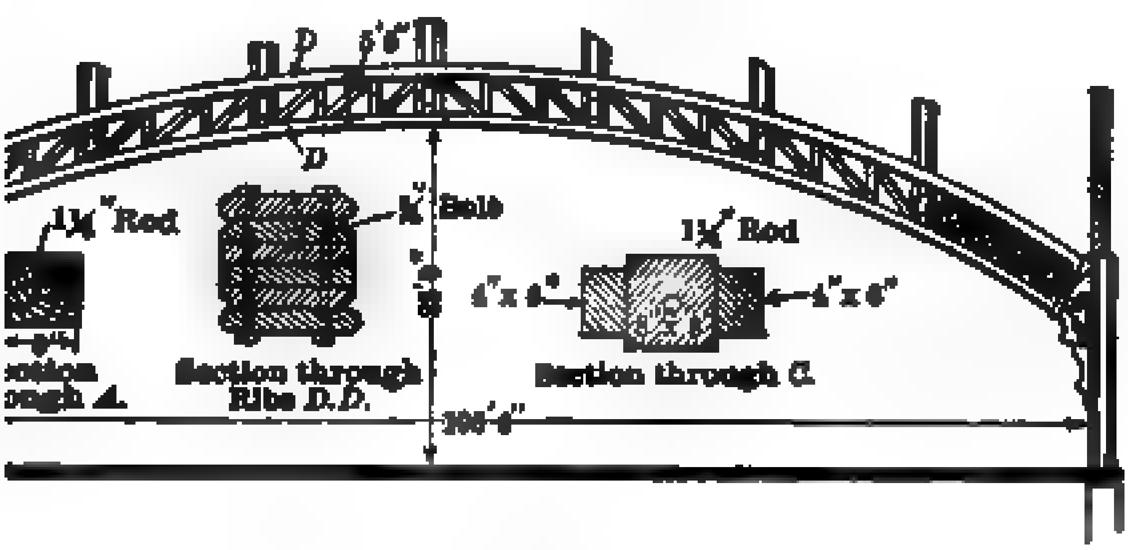


Fig. 38. Segmental Timber Arch

he Fifth Avenue Riding-School, New York City. The ting the roof of the Fifth Avenue Riding-School,* New York unusual and very ingenious; and as it is an excellent example of the arched form of truss, a brief description is added. The room, which is 106 ft 6 in long by 73 ft wide, is shown in Fig. 39. t free from columns, the entire roof being supported by two of which is shown in Fig. 38. The entire roofing is supported resting on these two large ones, each of the latter, however,

in 1905. The old trusses were used in the altered structure.

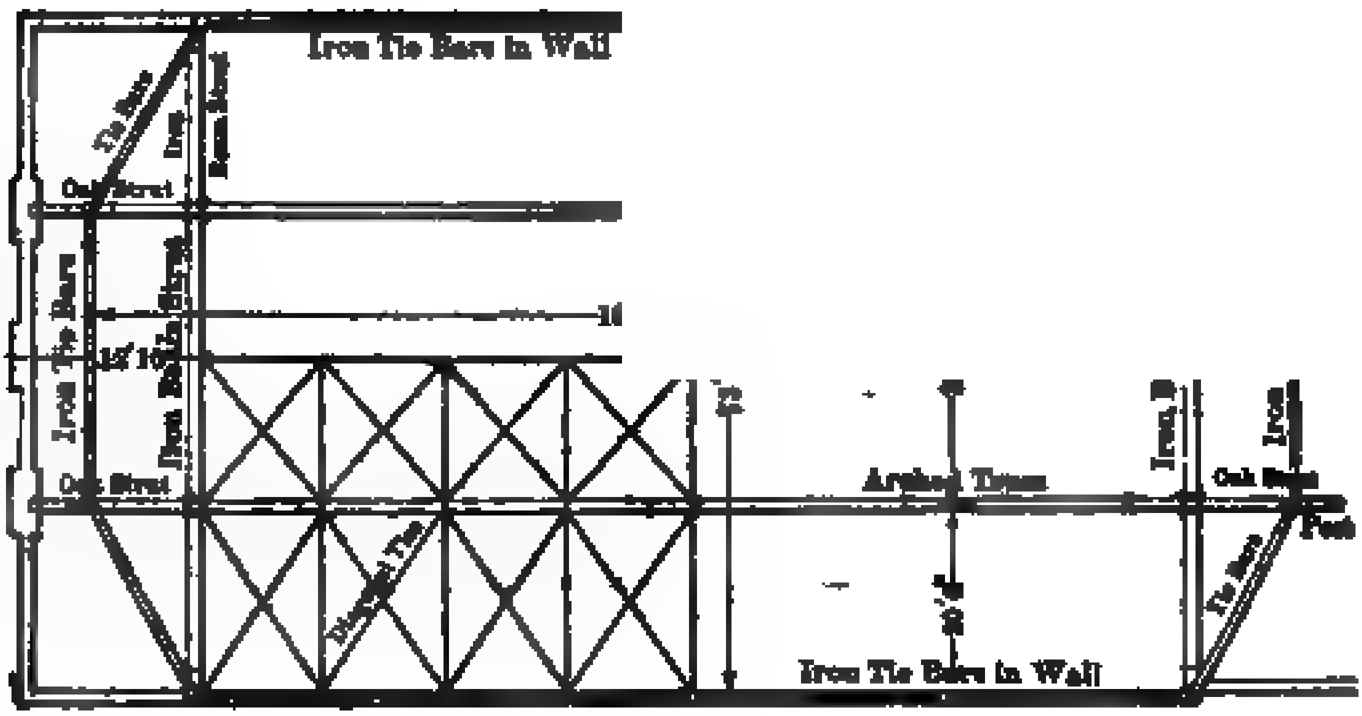
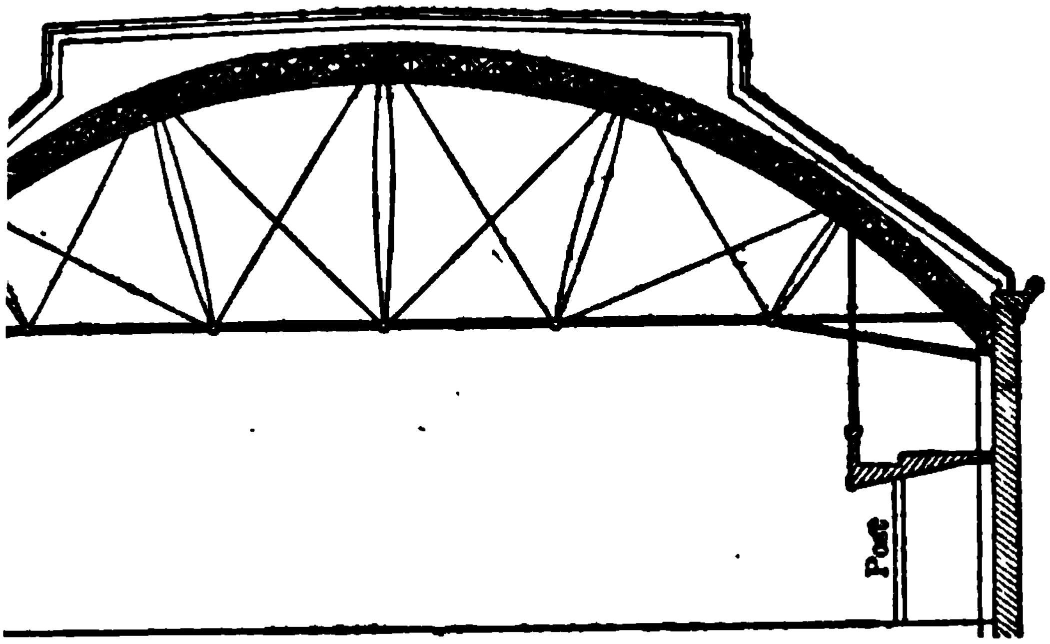


Fig. 39. Plan of Truss-framing of Fifth Avenue Riding-school, New York City

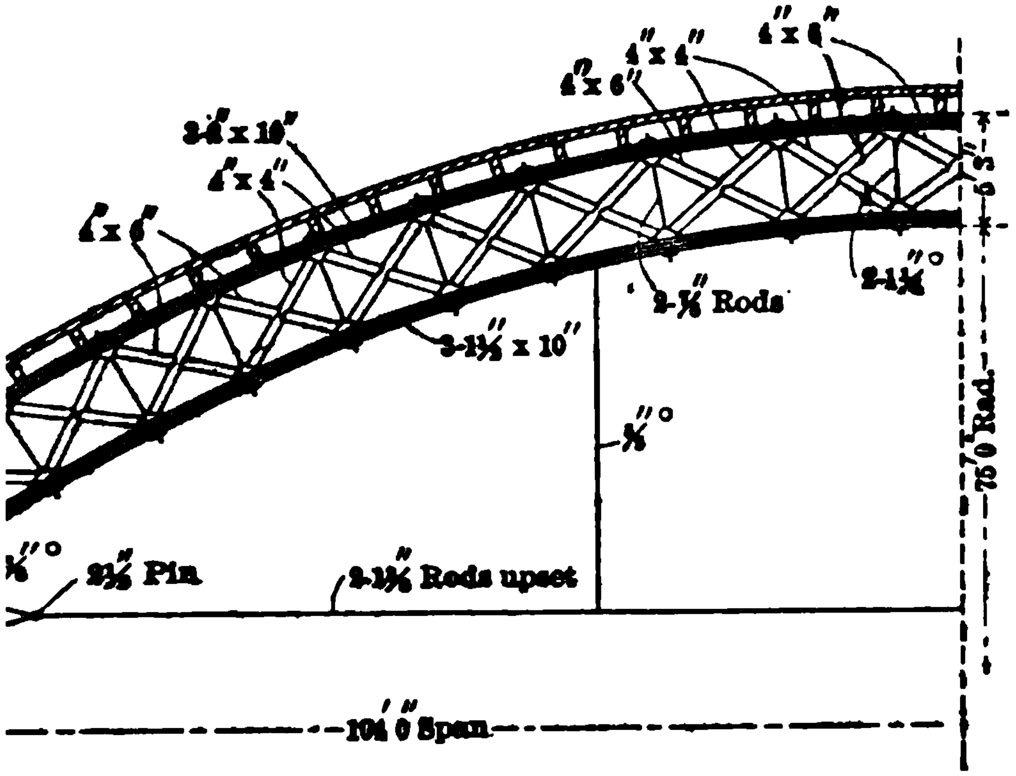
Fig. 40. Detail of Iron Skewback and Post of Truss Shown in Fig. 38

ying a roof-area, equal to about 2 930 sq ft, and a great amount
work. The method employed to resist the thrust of these large
the use of rods showing in the room is very ingenious. Opposite
of the iron posts which receive the arched ribs are oak struts



ed Wooden Truss. City Armory, Cleveland, Ohio. Span 79 feet

iron tie-bars and heavy iron beams and together forming a
t each end. These two trusses are prevented from being pushed
-in iron tie-bars in each side wall, as shown in the plan (Fig. 39).
the two iron posts are tied together by iron rods running under



Arched Wooden Truss, Sänger Hall, Philadelphia, Pa.

length of the room. Altogether this gives for the tie-rods
by 1-in iron bars and one 1½-in-diam iron rod, equivalent
tie-bars. Enlarged sections of the ribs, uprights and braces
8. It should be noticed that the uprights have iron rods
holding the two ribs together. Fig. 40 shows a detail, or

enlarged view, of the iron SKEWBACK and post at each end of the truss shown Fig. 38.

Truss for City Armory, Cleveland, Ohio.* Fig. 41 shows the meth adopted for supporting the roof and gallery, the arch being of wood.

Truss for Sänger Hall, Philadelphia.† Fig. 42 shows one-half of ARCHED WOODEN TRUSS which, with seventeen others, was designed to supp the roof over the central bay of Sänger Hall, Philadelphia, Haselhurst & Hud

Line

Fig. 43. Three-centered Curved Wooden Truss. O. N. G. Armory, Cincinnati, O

architects. This building was erected in 1897 for the use of the Eightee National Sängerfest, and was intended only for temporary use. With dimensions slightly increased, however, these trusses would be suitable permanent use. They were spaced 20 ft center to center. A descriptio the building and trusses was published in the Engineering Record of Januar 1897.

Truss for the O. N. G. Armory, Cincinnati, Ohio. Fig. 43 shows a t used in this building. The curve of the axial line of the arch-truss is a th centered ellipse. Hannaford & Sons were the architects of the building G. Bouscaren was the designer of the trusses. (See the Engineering and B ing Record, December 7, 1889.)

* The building has been remodeled and is now used for commercial purposes.
† This building was torn down immediately after the meeting.

2. Types of Steel Trusses

' Pitched Roofs. For ordinary conditions and for spans under
ıe of the types shown in Figs. 44 to 55 will generally meet the
f strength and economy. Trusses of these types are composed
s and angles
l joints. This
cheaper con-
ı combination
ods with pin-
o much more
ne dimension
es not exceed
hey can be
:ed up in the
e they are
dgment will
parts which
r rail, leaving

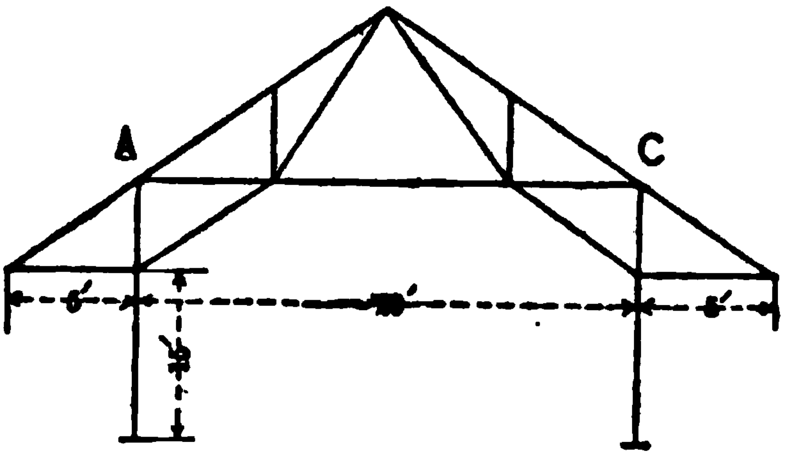
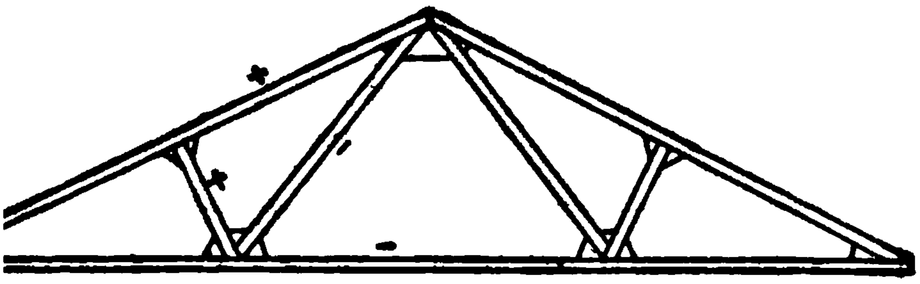


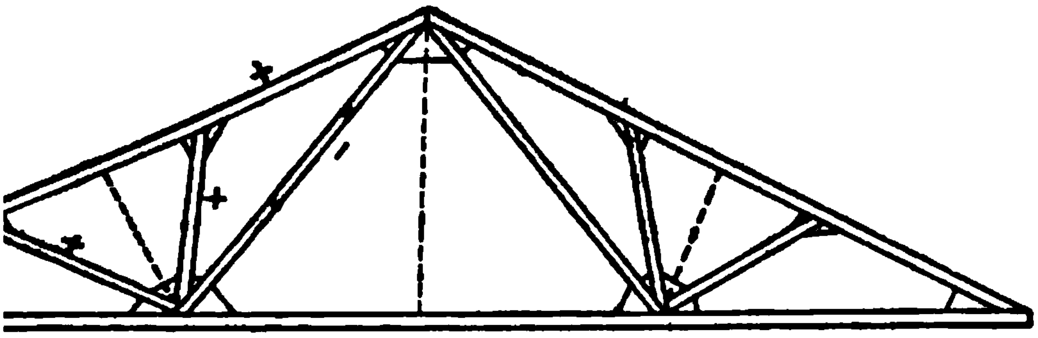
Fig. 44. Truss for a Narrow Shed or Shop

be riveted at the building; but entire trusses having spans even
raised from the ground and put in place. Occasionally a struc-
magnitude that this is not feasible, in which case the trusses
n parts and riveted afterwards. For a narrow shed or shop a



Simple Fink Truss. Spans from Twenty to Thirty-six Feet

e shown in Fig. 44 is the most economical, the truss proper
ı enclosed within the points *A*, *B*, *C*. This truss is practically
shown in Fig. 45. For spans of from 24 to 48 ft, and inclina-
ıg 6 in to the foot, the types shown in Figs. 46 and 47 are the
russes of the types represented by these two figures are called



16. Fan Truss. Spans from Thirty-six to Fifty Feet

e truss shown in Fig. 45 is known as a SIMPLE FINK TRUSS.
Fig. 47 is supported on columns, the KNEE-BRACES *B* and the
essed only when the building is subjected to wind-pressure.
by the middle dotted line, Fig. 46, is generally inserted.
ıstruction demands three purlins on each side of the truss,

one of the forms shown in Figs. 48, 49, 50, or 51 should be used. The te
FRENCH appears to be generally given to those trusses in which the tie-bea
raised or cambered in the middle. The truss shown in Fig. 51 may be ca

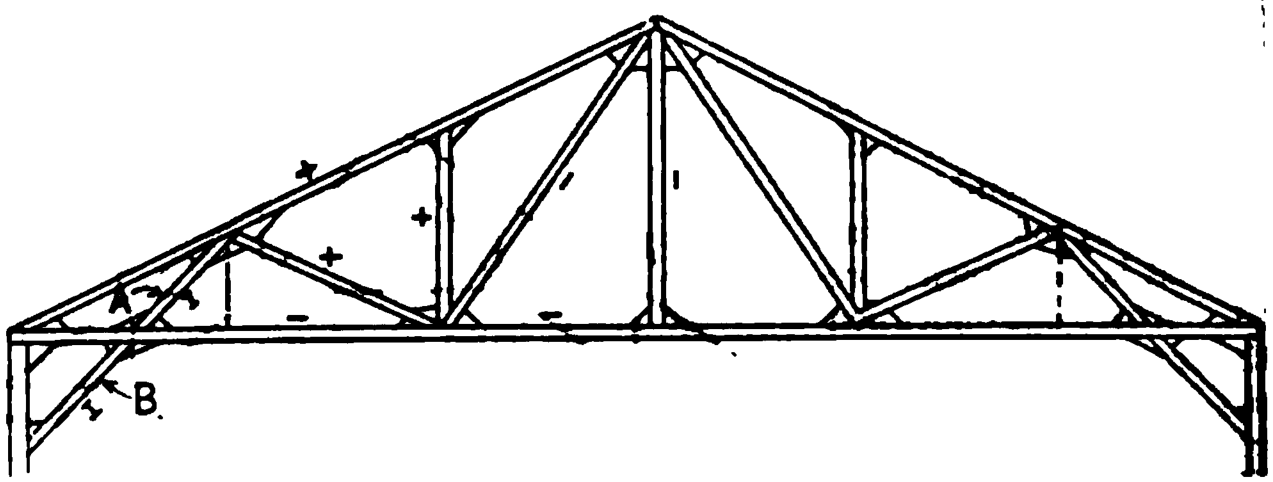


Fig. 47. Fan Truss with Knee-braces. Spans from Forty to Sixty Feet

a TRIANGULAR PRATT TRUSS as the web is composed of verticals in compressi
and diagonals in tension. This truss is not as economical as the FINK TRU
except when the inclination of the rafter is less than ¼ pitch. This is
account of the great length of the web-members in compression. In designi

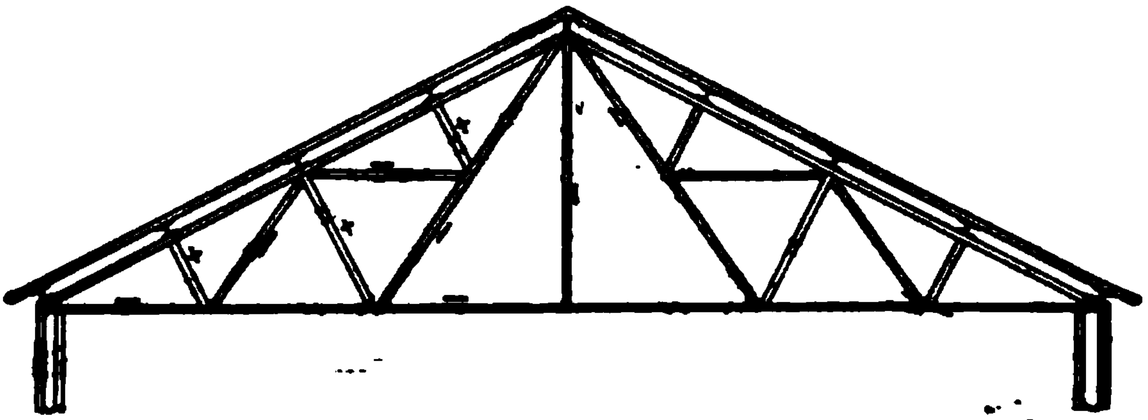


Fig. 48. Fink Truss. Spans from Forty to Eighty Feet

steel trusses it is desirable to have as many members, and especially as m
long members, in tension as possible, as a given weight of steel resists a m
greater stress when in tension than when in compression. The great econo
of FINK TRUSSES and FAN TRUSSES lies in the fact that most of the memb

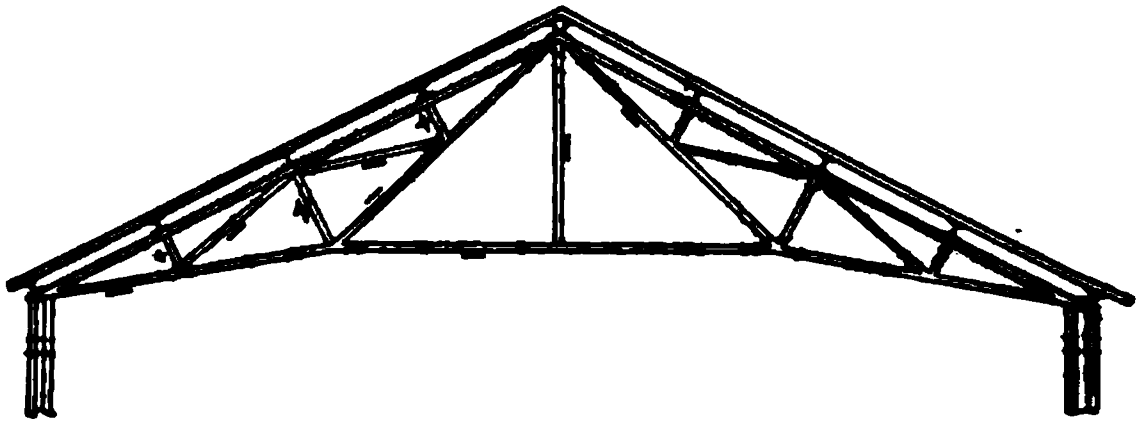


Fig. 49. French Truss. Spans from Forty to Eighty Feet

are in tension and the struts are short. By comparing Figs. 50 and 51,
seen that the inner strut in Fig. 50 is only one-half as long as the strut in
51. If the roof is hipped it is desirable to have vertical members in the
trusses to receive the short trusses or TRUSSED PURLINS.

nk and Fan Trusses. The DEPTH of these trus
y determined by the roofing-material. Thus slate sho
ı which the rise is not equal to one-third the span. I
should be not less than one-fourth and for corrugate
th the span. Steel-roll roofing may be used where
the span. There are many kinds of so-called READ
which may
y slope ex-
:o the foot.
'el roofing
used on a
⅝ in to the
ıg the con-
: roof and

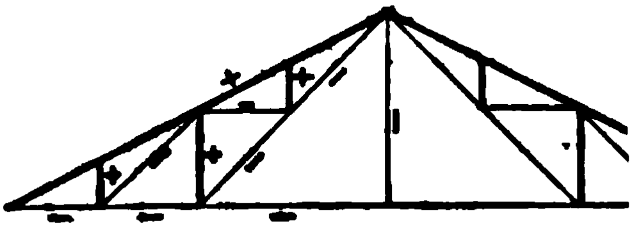
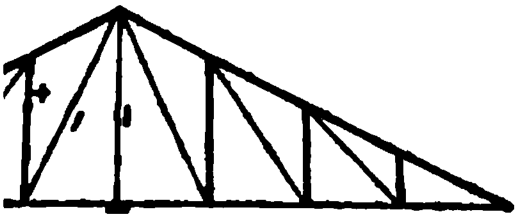


Fig. 50. Fink Truss with Vertical S

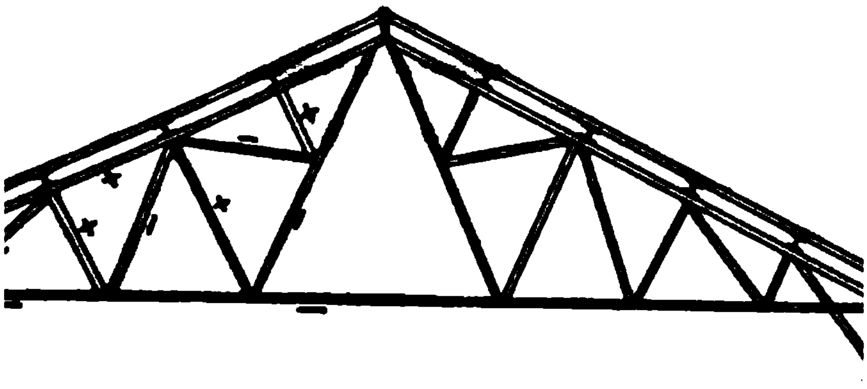
: trusses, the most economical pitch for a roof is a
or what is commonly called a QUARTER-PITCH, the
n for each 12 in of run, or 26° 34′. When the r
the span some other type of truss is generally
tion of the roof is determined almost entirely by th
ise is generally made from 6 to 7 in in 12 in. V
TRUSSES or FAN
having inclinatio
rafters not exceed
is more economical
a horizontal chord
truss whose bott
has a rise of 2 or
Fig. 49, presents



Triangular Pratt Truss

'er, than one with a horizontal chord. Raising t
ılly increases the stresses in the truss-members
For steep roofs, however, it is generally as eco
hord, because of the shortening of the members.

ıels. The NUMBER OF PANELS that should be us
determined in great measure by the construction c



Fink Truss with Knee-braces. Span Sixty-eight Feet

ırlins are used the length of a panel may be as gre
rafters and the planking of the roof is nailed dire
e placed not more than 8 ft apart; and if the roof
secured to the purlins, the purlins should be not
henever the purlins are more than 4 ft apart th
s-joints to prevent large bending-stresses in the

The spacing of the purlins, therefore, generally determines the number of pan in each half of the truss. For this reason also, the same form of truss may required for spans of 40 and 80 ft; but of course the members will not be heavy in the 40-ft truss as in the one with greater span. Most of the trus shown in Figs. 45 to 55 are drawn from executed designs and give a go idea of the most economical division for different spans.

Truss over Car-Barn, Newark, N. J. When stresses due to flexure developed in the truss-rafters, that is, when they are loaded between the joi

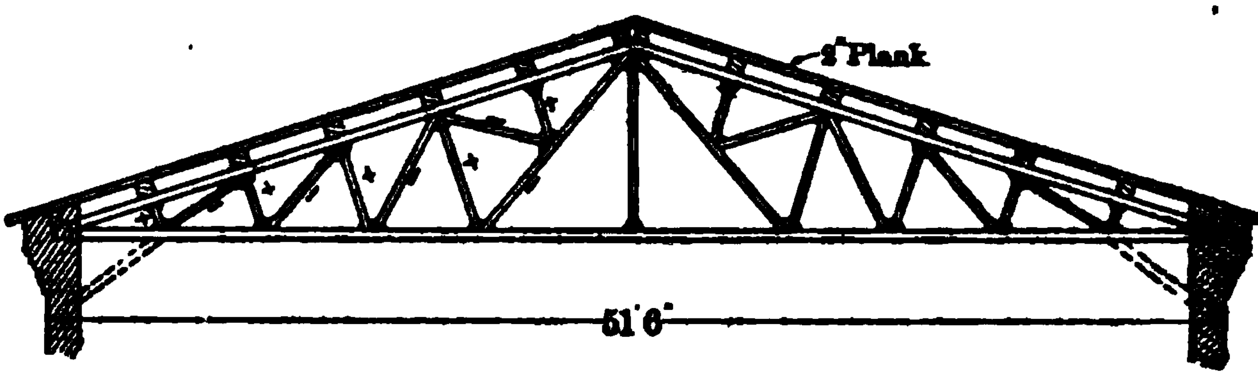


Fig. 53. Fink Truss. Span Fifty-one Feet Six Inches

the distance between the latter should not exceed 9 ft, and preferably 7 or depending somewhat upon the distance between the trusses themselves. diagram shown in Fig. 55 represents one-half of one of the steel trusses used roofing a car-barn for the North Jersey Railway Company, Newark, N. J. Th are 13 of these trusses spaced 19 ft 2¼ in on centers, each having a span 98¼ ft between the centers of the supporting columns, to which they are riv by splice-plates engaging the end connection-plates and the webs of the colum The dimensions of the principal members of these trusses are indicated in c

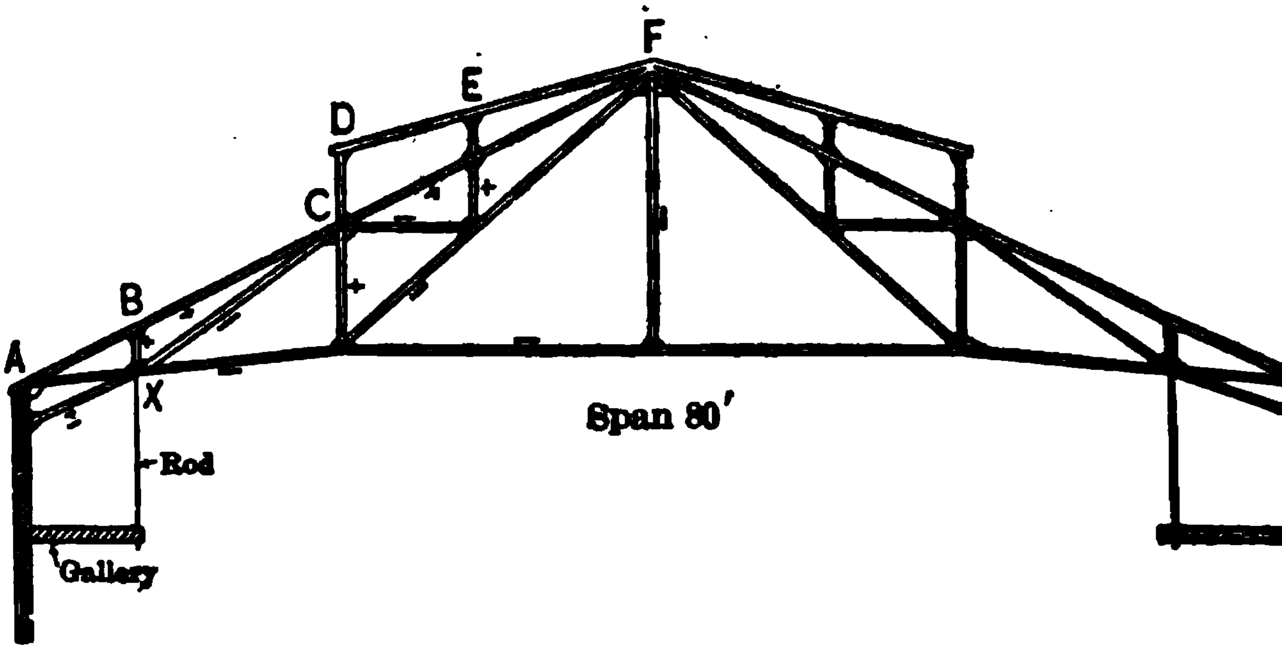


Fig. 54. Fink Truss with Vertical Struts, for Drill-hall. Span Eighty Feet

nection with Fig. 55. There is a more complete description in the Enginee Record of June 22, 1901. These trusses were shipped in four sections, w were assembled on the ground in a horizontal plane and riveted up comp The bottom chord was stiffened by rails lashed on each side of its e length, and a sling being attached to the apex of the top chord, the truss lifted and set on top of the columns by a gin-pole, 50 ft in length. roofing consists of corrugated iron supported by 5-in I-beam purlins, wei 10 lb to the foot, spanning from truss to truss and bolted to the rafters

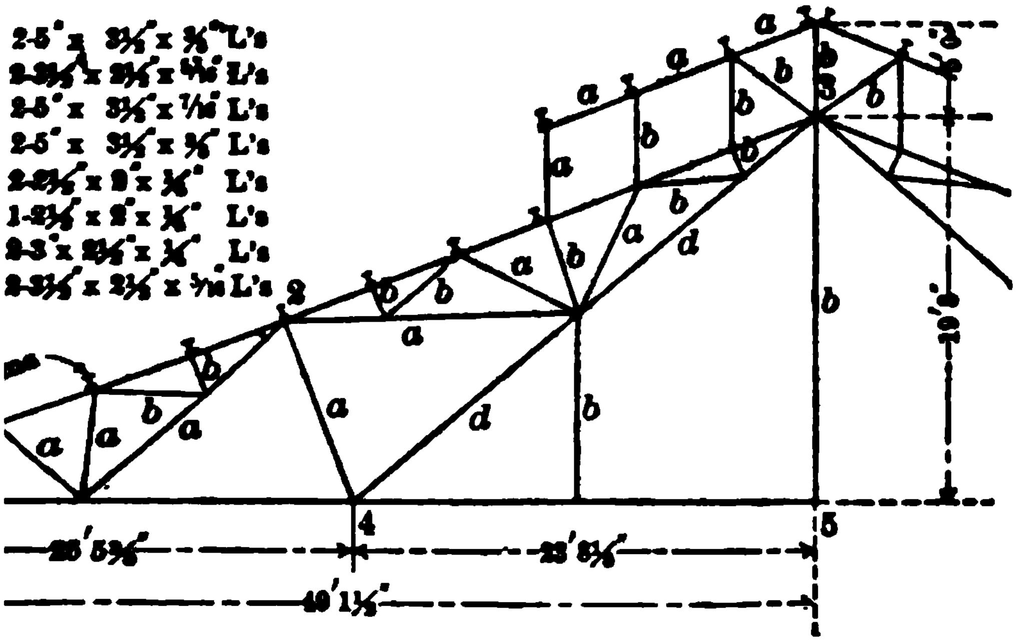
ch end. The general spacing of the purlins is 4 ft 9¾ in. This
le of an extremely light roof, the weight of each truss being about
he entire weight of truss, purlins, bracing of lower chord and
roofing being only 8 lb for each horizontal foot of surface cov-

List of Descriptions of Different Types of Roof-Trusses

Engineering Record

Date	Type	Number of panels
h 19, 1892	Howe	8
o, 1901	Fink	8
	Fan	12
ry 4, 1902	Fink	8
ary 22, 1902	Pratt	6
t 12, 1905	Fan	12
mber 2, 1905	Fan	12
mber 16, 1905	Fink	8
mber 2, 1907	Truss	16
nber 16, 1911	Truss	12
er 7, 1911	Fink	16

rill-hall. The truss shown in Fig. 54 was designed for the roof
ing a span of 80 ft and a spacing between trusses of 20 ft. The
onstructed with 2 by 8-in rafters supported by purlins at the



r Car-barn, Newark, N. J. Span Ninety-eight Feet, Three Inches.
(See, also, Chapter XXVIII, Fig. 25)

E and *F*. Sashes were to be placed in the rise *CD*, to light
building. The joint at *X* was located with reference to the
ery-rod; but if there had been no gallery it would have been
space the vertical struts uniformly, as in Fig. 50. In all
ted the PLUS SIGN adjacent to a member denotes that the
RESSION, while the MINUS SIGN denotes that it is in TENSION.
e the main rafter, as *CD*, *DE* and *EF*, in Fig. 54, and *a* and *b*
rm a part of the truss proper, but are merely a framework to

support the elevated roof, and in drawing the stress-diagram for the verti loads they would be omitted.

In the issues of the Engineering Record given in Table III may be found scriptions and illustrations of several types of roof-trusses, including the fo described above.

Fink Trusses with Pin-Joints. The use of PIN-JOINTS in ordinary r trusses has practically been abandoned, even for long-span heavy trusses. the Engineering Record of March 12, 1892, there is a description of a Fink tr with pin-joints. The truss is heavy and is built entirely of rolled metal. T tension-members are 5, 6 and 7-in eye-bars. The span is about 105 ft.

Trusses for Flat Roofs. For supporting flat roofs or roofs having a not exceeding 1 in to the foot, one of the types shown in Figs. 56 to 60 will g

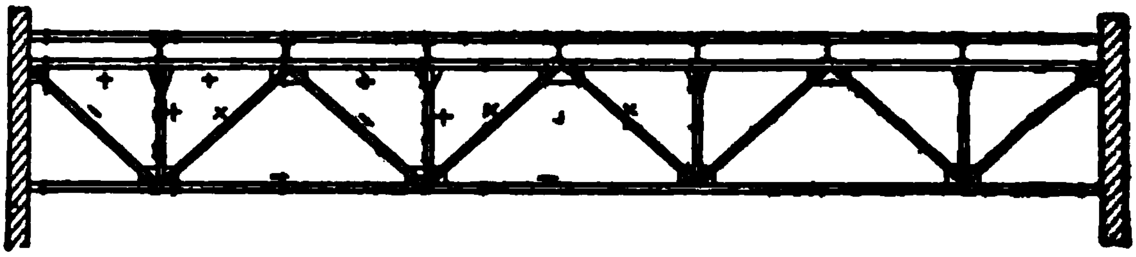


Fig. 56. Warren Truss with Verticals. Span Fifty-six Feet

erally be found economical, the choice of the particular type depending somew on the span and on whether the truss is supported by columns or by brick stone walls. For spans up to about 50 ft, either of the forms shown in Figs or 57 answer all practical requirements. The truss shown in Fig. 56 is inten to be used where the slope of the roof is at right-angles to the truss. It can built, however, with the top chord inclined as in Fig. 57. The end-diago in Fig. 56 are in tension, while in Fig. 57 they are in compression. The porti of the lower chord between the end-joints and the walls (Fig. 56) have no st

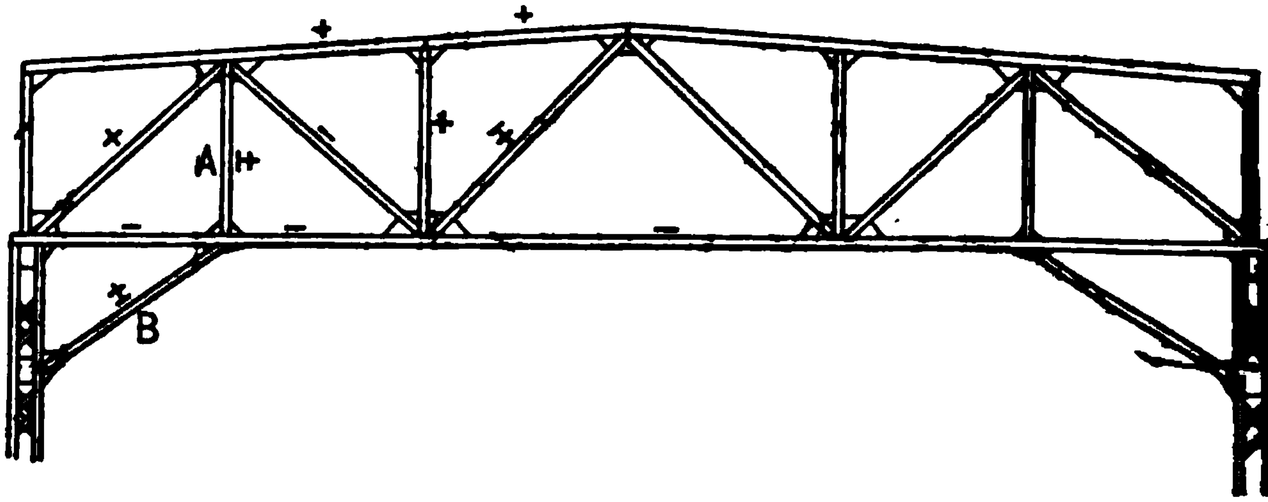


Fig. 57. Warren Truss with Verticals and Knee-braces. Spans from Thirty t Fifty Feet

from the roof-load, but are put in to add rigidity to the construction as a wh In trusses supported by brick walls this type is preferable to that shown in Fig while the latter is more suitable when the roof is supported by columns. vertical *A*, Fig. 57, is inserted to receive the tension or compression from *B*, and has no stress from the roof-loads.

Double Warren Truss. The truss shown in Fig. 58 is known as a DOU WARREN TRUSS, and is desirable where it is important to make the truss shallow as practicable. It can be built with light members, and is a very

specially suitable for roofs supported by steel columns. Fig. 58 a truss in actual use. The member in the middle indicated by the ould never be omitted, although examples may be found where it included. Fig. 59, also, represents a roof-truss which was con- a span of 57 ft and supported by steel columns. The entire load s transmitted to the columns at the intersection of the diagonals

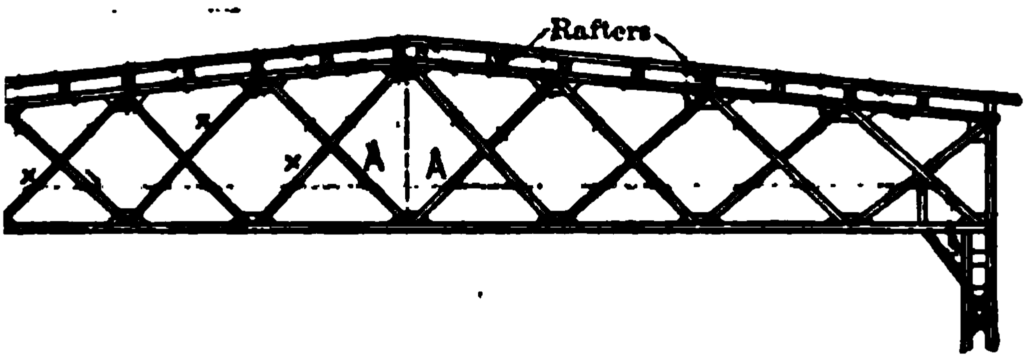
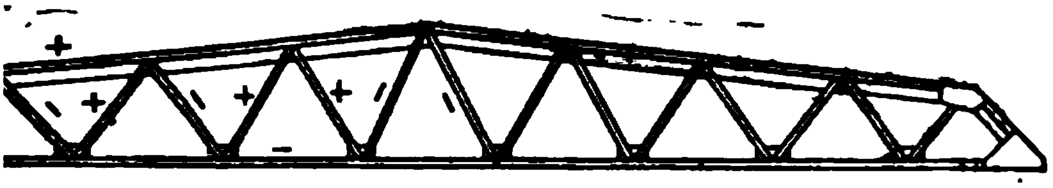


Fig. 58. Double Warren Truss

p chord. Fig. 60 shows a truss of 96-ft span over a pier-shed, y, the trusses being spaced 20 ft apart. They are about 10 ft 1 300 lb each. They were delivered from the shops completely riveted, and were raised and set in position by falls suspended s. The dimensions of these trusses are given in the Engineering ary 18, 1896.

Fig. 59. Pratt-truss Type. Span Fifty-seven Feet

d **Minus Signs** in these illustrations, as has been mentioned COMPRESSION and TENSION, respectively, under a uniformly dis- ad. The PLUS and MINUS SIGNS used together indicate that y be subject to EITHER TENSION OR COMPRESSION according of the wind or to the manner of distribution of the snow. In russes unsymmetrical loads may change the stresses in the



-truss Type. Pier-shed, New York City. Span Ninety-six Feet

e middle of the truss. This CHANGING OF STRESSES due to is considered on pages 1096 to 1104. The trusses shown in e almost invariably built with riveted connections and with hapes for all members.

ss, shown in Figs. 61 and 62, is the form of STEEL TRUSS support floor-loads, the members indicated by double lines ION and those indicated by single lines in TENSION. When

supporting floors are subject to moving loads, COUNTERTIES should be inserte where indicated by dotted lines. For trusses of this type PIN-CONNECTIONS a generally employed and are preferable to RIVETED CONNECTIONS.

The Quadrangular Truss. The truss shown in Fig. 63 is known as a QUA RANGULAR TRUSS, and has the proportions of the truss over the amphitheat

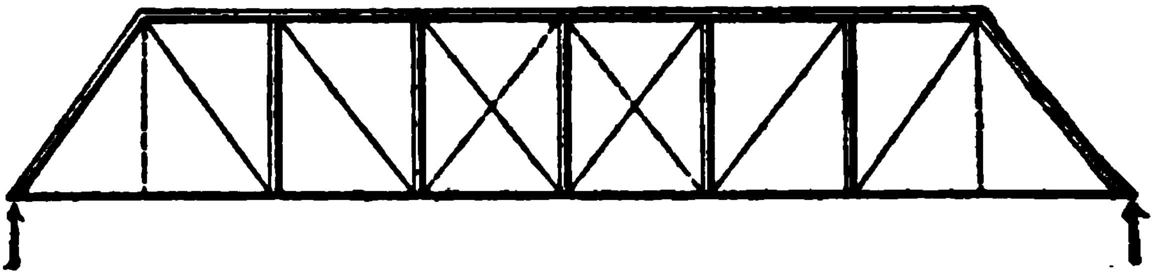


Fig. 61. Pratt Truss

of the Madison Square Garden, New York. Figs. 64 and 66, also, show variatio of this type, differing, however, from the latter in having all the diagonals each half-truss inclined in the same direction. In the typical truss their directi is usually reversed at about the middle of each half-span in order to keep the in tension. The PLUS AND MINUS SIGNS indicate the kind of stress produced

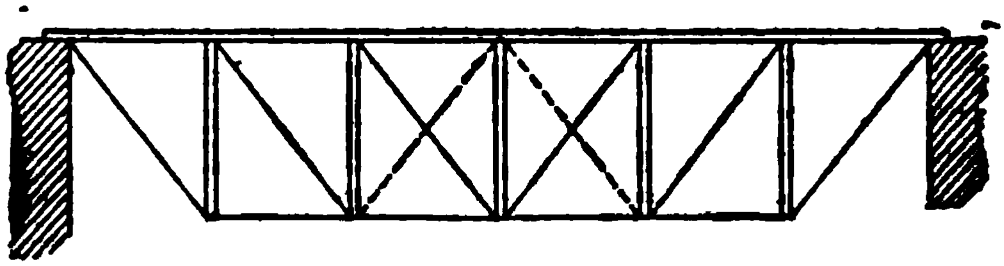


Fig. 62. Suspended Pratt Truss

a member by a uniformly distributed dead load. It should be noticed that t middle diagonals of trusses 64 and 66 are in compression. These trusses a well adapted to steel construction and to spans up to 180 ft. When the sp exceeds 100 ft one end of the truss should be supported on ROLLERS to allow f the EXPANSION or CONTRACTION in the steel. In these trusses the load is tra

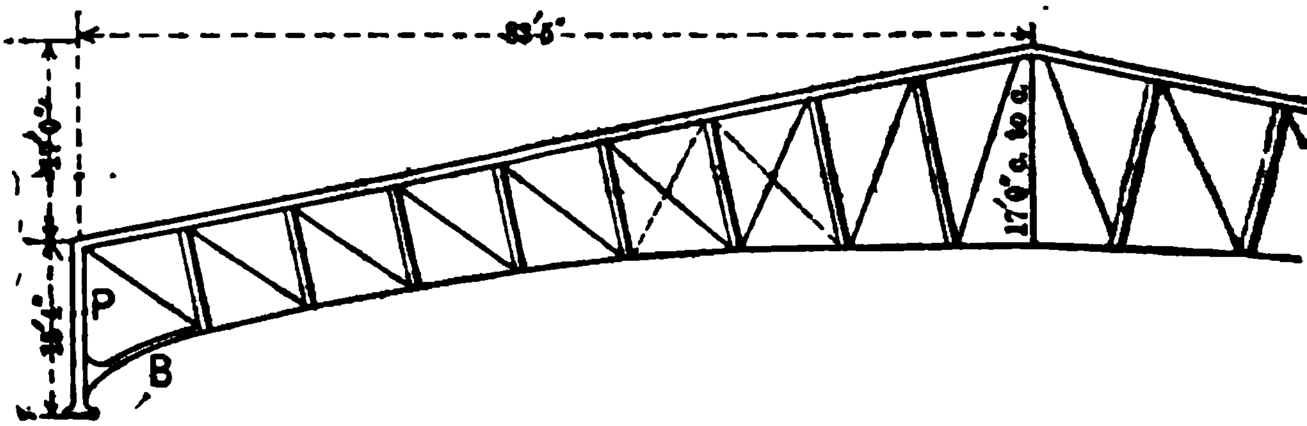


Fig. 63. Quadrangular Truss. Amphitheater, Madison Square Garden, New Yo City

mitted to the top of the column-support, the truss proper being included wit the points *A*, *B*, *C*, *D* and *E*, Figs. 64 and 65. The continuation of the bott chord to the columns is for the purpose of bracing the roof from the latter, th being no stresses in these end-chord members due to vertical loads. This me ber *B*, Fig. 63, and the corresponding member in Figs. 64 and 65 should be co structed to resist both tension and compression. For short spans the lo

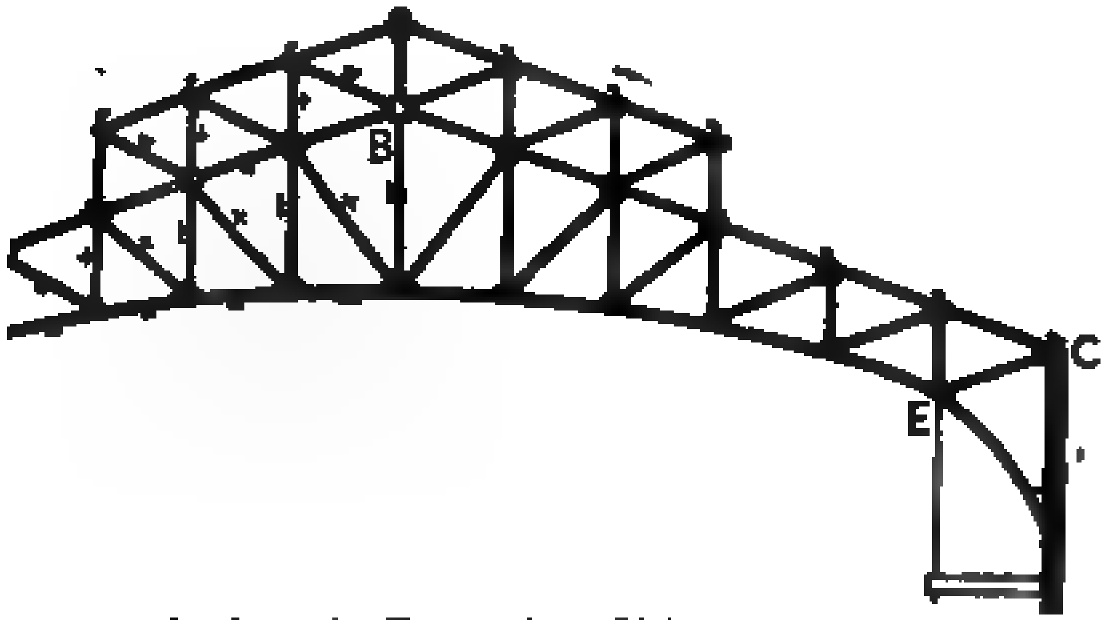
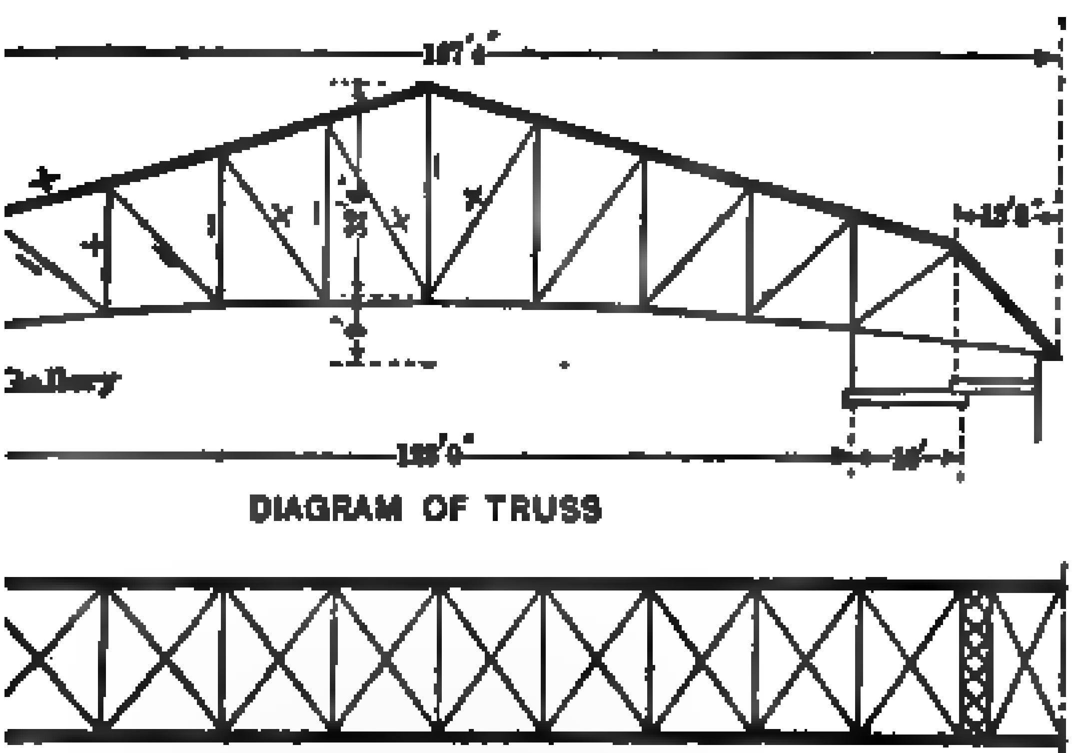


Fig. 64. Quadrangular Truss. Span Eighty Feet

Fig. 65. Quadrangular Truss



f Trusses in Auditorium, Kansas City, Mo. Plan of Two Trusses Showing Lateral Bracing

chord may be made in the shape of a semicircle or half-ellipse so as to g
more of an arch-effect. There are numerous examples in this country of q

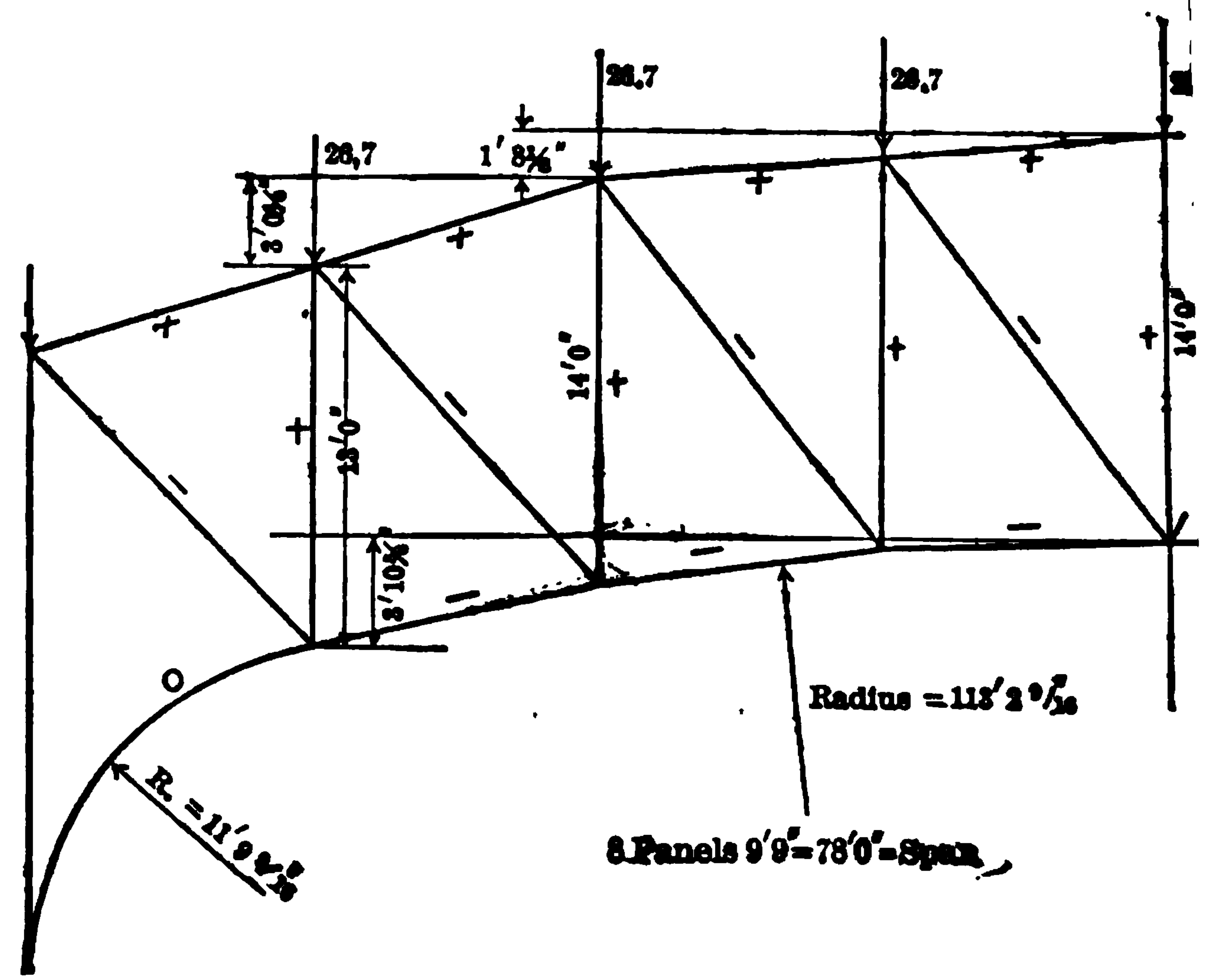


Fig. 67. Riveted Truss with Broken Top Chord. Power-house, Interborough Rap
Transit Company, New York City

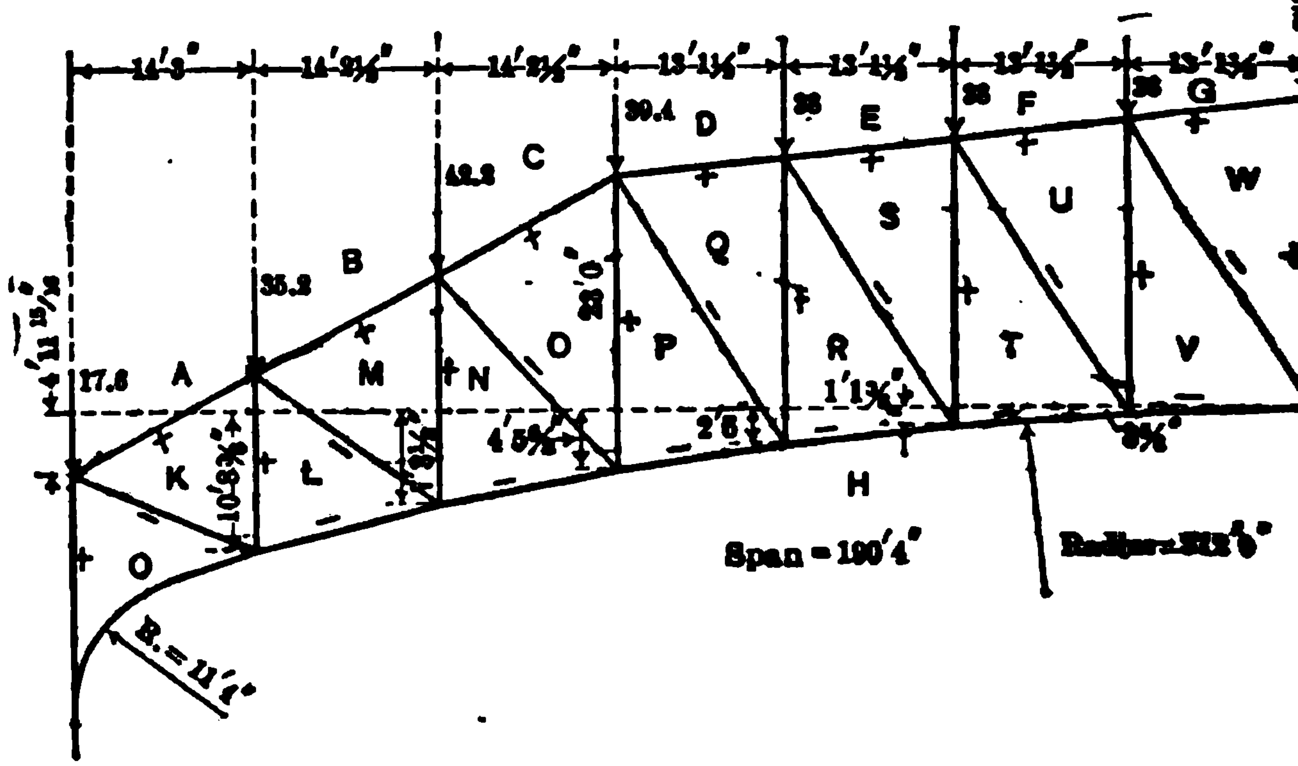


Fig. 68. Pin-connected Truss Over Drill-hall, 71st Regiment Armory, New York C

rangular trusses having spans of from 100 to 180 ft. For the wider spans i
customary to build the trusses with PIN-CONNECTIONS, EYE-BARS being used
the ties. When this is done it is usually necessary to insert COUNTERBRA

of each half of the truss as shown by the dotted lines, Fig. 63, asymmetrical or wind-load the stresses in the diagonals are gener- For spans less than 100 ft, the trusses may be built with RIVETED In this case the diagonals are generally made of angles capable oth tension and compression, the counterbraces, therefore, not . For this type of truss the stresses due to wind and snow should ndependently of the dead load and the members computed for stresses produced by every possible combination of loading.

r the Auditorium, Kansas City, Mo. A description, with the truss shown in Fig. 66, which is a diagram of one of the trusses is City Auditorium, may be found in the Engineering Record for and in the Engineering News of November 2, 1899.

uss with Broken Top Chord. A description is given in the ecord of October 15, 1904. The span is 78 ft between centers ng columns (Fig. 67).

)rill-Hall, New York City. A pin-connected truss, over the : 71st Regiment Armory, New York City, has a span of 190 ft escriptions of it are given in the Engineering News of June 16, : Engineering Record of July 2, 1904 (Fig. 68).

6. Arched Trusses

Between an Arched Truss and a Trussed Arch. For sup- f of very large spaces such as drill-halls, riding-halls, railway ., trusses in the form of arches or arches composed of trussed en employed. The essential difference between an ARCHED TRUSS RCH is that under vertical loads the supporting forces of an arched al, while for 1 they are

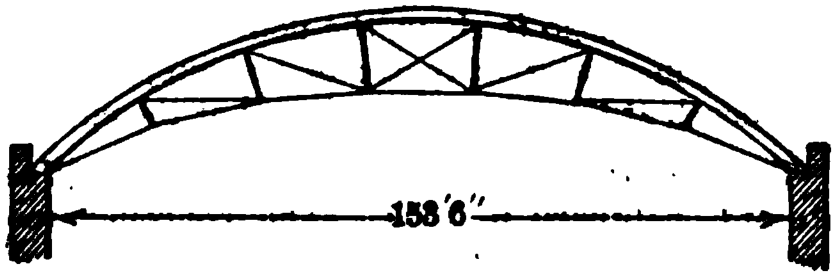
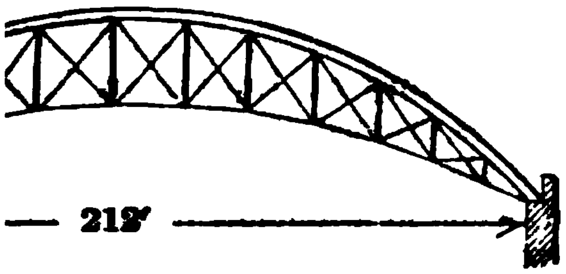


Fig. 69. Bowstring Truss

usses. Pre- r 1880 most iron trusses ere built in bow, from BOWSTRING russes of this type were built with spans of from 88 to 211 e at the middle of from 1/8 to 1/4 the span. At that time this type was considered the most economical for spans exceeding 120 ft, but in recent years they have been comparatively little used. Fig. 69 is the diagram of a bowstring truss with a span of 153 ft 6 in. The trusses in this particular case are spaced 21 ft 6 in apart.



70. Bowstring Truss

chord consists of a wrought-iron deck-beam 9 in deep, with plate, riveted to its upper flange. Towards the springing gthened with 7 by 5/8-in plates riveted on each side of the : struts are wrought-iron I beams 7 in deep. The bottom tional area of 6½ sq in and each diagonal tension-rod a

diameter of 1¼ in. Each truss is fixed at one end and rests on ROL at the other, allowing free expansion and contraction due to changes of t perature in the metal. Fig. 70 shows a similar truss having a span of 21 It consists of BOWSTRING PRINCIPALS spaced 24 ft apart. The rise is one- the span, the middle of the bottom chord rising 17 ft, and of the top chord 40 above the springing. The top chord is a 15-in wrought-iron I beam and bottom chord a round rod in short lengths, 4 in in diameter and thickene the joints. The ties of the bracing are of plate iron from 5 to 3 in in width, ⅝ in thick. The struts are formed of bars having the form of a cross. Du the last ten or twelve years a number of roofs have been supported on tru which can hardly be classed as SIMPLE TRUSSES; and yet it is questionabl they are TRUE ARCHES. Probably the frames act partially as simple tru and partially as arches.

Trusses for the Conservatory Building, Garfield Park, Chicago, I Engineering News, August 27, 1908. The roof is supported by POINTED TRUSS spaced 12 ft 6 in on centers. The truss-span is 80 ft 6 in, center to center end-supports. The chords of the trusses are parallel and connected by WARR BRACING. Both ends of the trusses are bolted to the supports and consequent there must be some horizontal thrust under certain conditions. The truss are riveted at all joints and have no HINGES or PINS.

Trusses for the Chicago and North Western Railway Station, Chicag Ill. Engineering Record, June 18, 1910. The roof over the main waiting-roo is carried by trusses each having a span of 90 ft 4 in and a rise of 31 ft and bei riveted to columns about 27 ft 6 in apart. All connections are riveted. T clear height of the bottom chords at the middle is 84 ft.

Trusses for the Peoria and Pekin Union Railway Trains-Shed, Peori Ill. Engineering Record, December 8, 1900. The trusses are riveted columns about 30 ft above the floor and spaced 20 ft apart. The truss-span 109 ft 4 in, center to center of end-supports, with a clear rise of about 10 The depth at the middle is 18 ft and at the end 6 ft. All connections are rivete

Trusses for the New Union Station, Washington, D. C. Engineeri Record, February 6, 1904. The concourse-roof is supported by CRESCE TRUSSES, each having a span of 132 ft 5½ in and a clear rise of 22 ft 5½ in. Th are spaced about 39 ft 4 in apart. One end of each truss rests upon maso and the other is riveted to a heavy plate girder. All connections are rivete The bottom chord at the middle is 45 ft above the floor. The trusses over t waiting-room of the same station have a span of 137 ft 8 in and a rise of 45 5 in. The chords are parallel and the ends are anchored with bolts to t masonry.

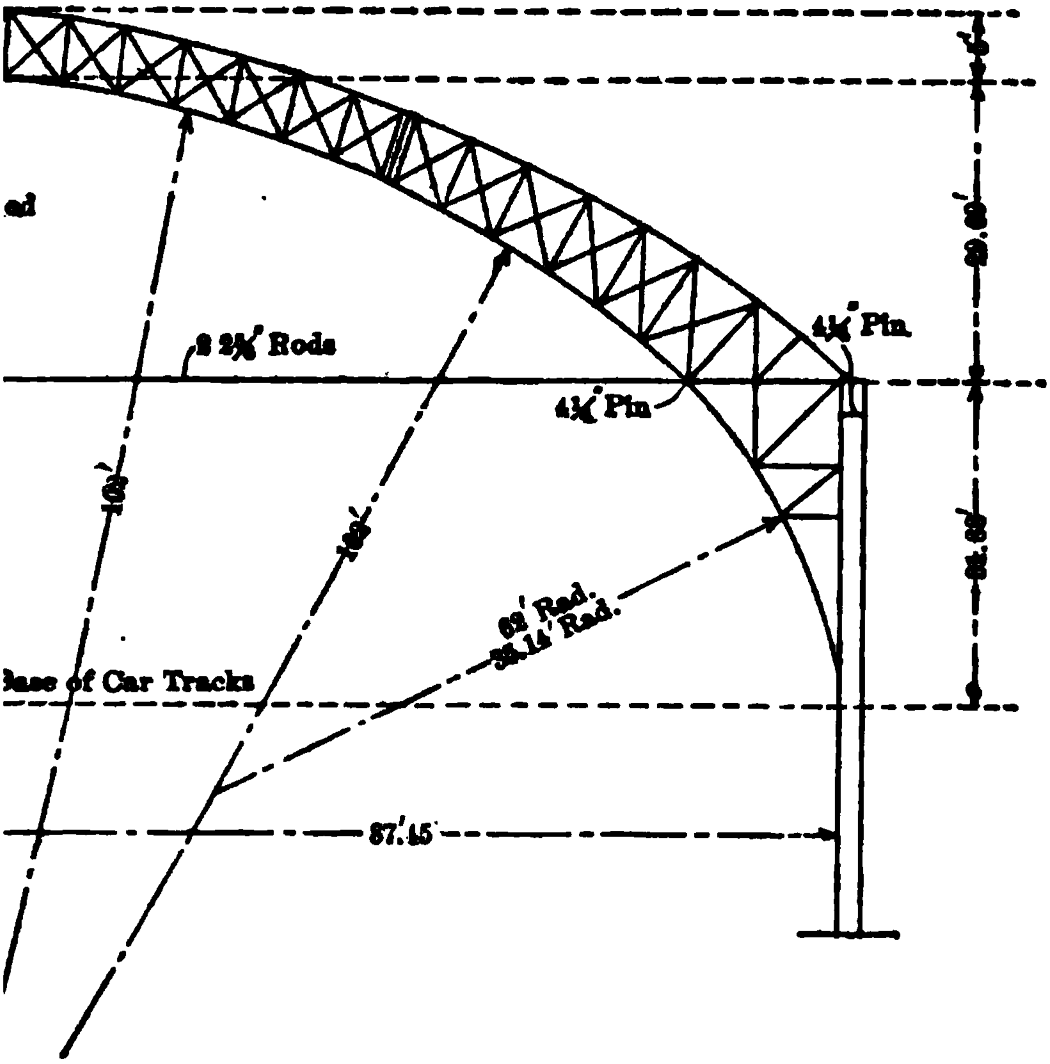
Trusses for the Riding-Hall, Armory for Squadron C, National Guar Brooklyn, N. Y. Engineering News, August 29, 1907. The main trusses ha a span of 179 ft 2 in and a rise of about 66 ft in the clear. The total depth the truss at the middle is 14 ft, while at the ends, where the chords appro each other and finally become vertical, it is 3 ft 3 in. One end is anchored the masonry and the other is on rollers. The trusses are in pairs 10 ft 11½ on centers and the pairs are spaced 38 ft 8½ in on centers. All connections riveted.

Trusses for the New Rock Island Terminal Station Train-Shed, Chica Ill. Engineering Record, September 12, 1903. Engineering News, August 1903. The trusses over the tracks have a span of 221 ft 1 in center to cent the end-pins, a rise of 28 ft and a depth at the middle of 25 ft 6 in. They

columns and are spaced from 10 ft 3 in to 19 ft 6 in apart. All
ections are made with pins.

russes with Horizontal Ties. CURVED OR ARCHED TRUSSES are
:ted with a horizontal member connecting the ends at the supports.
he structure as a whole, including the horizontal member, usually
:-ROD, a SIMPLE TRUSS requiring only vertical supporting forces
ads, provided one end is free to move, as it is when placed on
1 the trusses are supported by long columns it may be assumed
have freedom. A few examples are given, some of which are
sed as TRUE ARCHES.

r the Sullivan Square Station, Elevated Railway, Boston,
ieering Record, June 15, 1901. Fig. 71. These ARCHES spring



Truss for Sullivan Square Station, Elevated Railway, Boston, Mass.

1s and are provided with tension-rods which take up the thrust.
:ests on two 4¼-in pins at each end, as indicated in the diagram,
; connected to them. The bracing below each pin is riveted
d the arch itself is built of angles and plates with riveted con-
1A shows the joint at *A* where the tie-rods are connected and
SUSPENSION-ROD from the crown of the arch. This construc-
1 principle as that of the WOODEN ARCH shown by Fig. 42.

Express Company's Receiving Station, New York City.
ord, October 22, 1904. The roof-trusses in this building are
n brick walls at the level of the second-story floor and have
ted by I beams which form a part of the floor-framing of the
ch truss has a span of 74 ft 4 in and a clear rise of 27 ft. They
24 ft 5 in apart and have all connections riveted. Since the

es are very heavy one might be led to classify these trusses with TRUE ARCH xed at the ends; but as the condition of FIXED ENDS rarely obtains in practi : is better to consider this type of structure as an ARCHED TRUSS with a tie-r r possibly as similar to the type shown in Fig. 75.

Table IV. General Dimensions of a Few Three-Hinged Arches

Location	Span ft	Span in	Rise ft	Rise in	Tie
Syracuse University	101	4	...	...	Floor-beams
Lawson Riding-Academy	106	0	56	2¼	No tie
Machinery Hall, Chicago Exp.	121	10	96	3	†Two 1⅞ × 1⅛
22nd Reg. Armory, New York	134	0	32	6	
Coliseum, Chicago (new)	149	9	66	6	†2½ × 2½
Newark, N. J., Armory	163	6	73	5⅝	†Two 2¾ round rods
Government Bldg., St. Louis Exp.	172	0	69	9½	9-in I beam
Coliseum, St. Louis	178	6	80	0	No tie
Hartford, Conn., Armory	181	0	90	2½	No tie
Frankfort, Germany, Train-Shed	184	0	94	(about)	No tie
69th Reg. Drill-Hall, New York	189	8	103	4½	†Two 1¾-in round rod
5th Reg. Armory, Baltimore, Md.	190	4	88	0‡	Two channels
47th Reg. Armory, Brooklyn, N. Y.	191	4	84	0	†Two 4 × ⅞-in plates
Coliseum, Chicago (old)	215	0	72	0	
74th Reg. Armory, Buffalo, N. Y.	227	0	94	0	
Coal-Shed, Wende, N. J.	230	0	...	...	†9 × 7/16-in plate
Jersey City Train-Shed	252	8	89	9¾	Two 12-in I beams
Philadelphia Train-Shed	259	0	88	3½	
Broad Street Station, Phila.	300	8	100	4	
Manufactures and Liberal Arts Bldg., Chicago Exp.	368	0	206	4	

Location	Distance, center to center*	Reference
Syracuse University	17 ft 11½ in	R. Aug. 22, 1908
Lawson Riding-Academy	32 ft 0 in	R. Dec. 31, 1904
Machinery Hall, Chicago Exp	50 ft 8 in	R. Dec. 24, 1892
22nd Reg. Armory, New York	11 and 52 ft	N. May 5, 1910
Coliseum, Chicago (new)	22½ to 25 ft	N. Sept. 14, 1899
Newark, N. J., Armory	31 and 26½ ft	R. May 26, 1900
Government Bldg., St. Louis Exp.	35 ft 0 in	N. Sept. 29, 1904
Coliseum, St. Louis	36 ft 8 in	N. Aug. 10, 1899
Hartford, Conn., Armory	6 and 52⅔ ft	R. Sept. 12, 1908
Frankfort, Germany, Train-Shed	33 ft 6 in	R. Mar. 5, 1892
69th Reg. Drill-Hall, New York	6⅝ and 38¾ ft	R. June 3, 1905
5th Reg. Armory, Baltimore, Md.	....	R. May 14, 1904
47th Reg. Armory, Brooklyn, N. Y.	34 ft 4 in	R. Dec. 23, 1899
Coliseum, Chicago (old)	46 ft 8 in	N. Nov. 12, 1896
74th Reg. Armory, Buffalo, N. Y.	....	R. June 9, 1900
Coal-Shed, Wende, N. J.	22 ft 10¼ in	R. Oct. 3, 1908
Jersey City Train-Shed	14½ and 43½ ft	N. Sept. 25, 1899
Philadelphia Train-Shed	....	R. July 16, 1892
Broad Street Station, Phila.	....	R. June 10, 1893
Manufactures and Liberal Arts Bldg., Chicago Exp.	....	N. Sept. 1, 1892

* Center to center of end-supports. † Dimensions in inches.
‡ To lower chord. N. Engineering News. R. Engineering Record.

or Drill-Hall, 13th Regiment Armory, Scranton, Pa. Engi-
ord, August 24, 1901. These roof-trusses are about 5 ft deep and
out 12 ft on centers. The truss-span is 156 ft, over all, with a rise
e clear. The ends rest on
d are connected by a tie
two 1⅝-in round rods.
motion is provided at one
g the holes for the anchor-

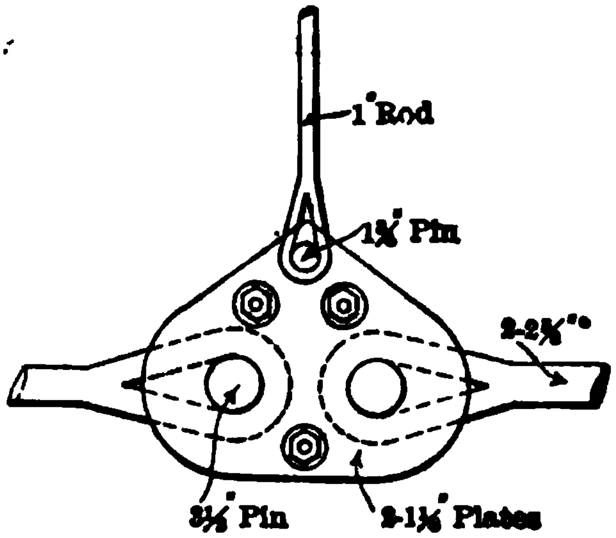


Fig. 71A. Detail at *A*, Fig. 71.

for Armory Drill-Hall,
R. I. Engineering
l 13, 1907. The type of
d in this building is com-
a THREE-HINGED ARCH,
pin at each support and
wn; but the two end-pins
by a tie and one end-shoe
ith rollers and hence the
, SIMPLE TRUSS composed of three members, two of which are
nselves. The truss-span is 166 ft 8 in and the rise about 61 ft.
e riveted and spaced about 26 ft 1 in on centers.

r the Pennsylvania Railway Train-Shed, Pittsburgh, Pa.
Record, August 23, 1902. The trusses have three HINGES and a
LER-BEARING at one end. The truss-span is 255 ft ⅝ in between
s, the rise 93 ft between pin-centers and the depth at the center
sses are riveted and stand in pairs 9 ft on centers and the pairs
ft 6 in on centers.

-hinged Arch as employed for supporting the roofs over large
eds, drill-halls, etc., is composed of two CURVED TRUSSES, usually
rm and dimensions, resting upon PINS at the supports and con-
N over the middle of the span. The supports are assumed to be
on and are often connected by a TIE to insure stability and take
tal thrust of the arch. While a metal tie between masonry sup-
make these supports FIXED in position under all or any conditions
for all practical purposes they may be so considered; and these
STRUCTURES which have ties, provided there is no arrangement
nd-movement due to roller-bearings, etc., may be classified with
pports must resist all horizontal as well as all vertical forces.
ns are usually placed below the floor-level so that the tie-rods,
y be concealed by the floor or even made a part of its framing.
onditions the arches can be so designed that the horizontal thrust
all and the supports designed without the use of the horizontal
al advantages of the THREE-HINGED ARCH for the class of build-
ationed are economy and a maximum amount of clear space
ss. Much of the economy results from the omission of support-
The base of the arch being very near the ground-level, it is also
o resist wind-pressure. Another advantage of this type is the
allowed under temperature-changes without causing additional
nembers of the structure, the middle part rising or falling freely
tation of the half-trusses about the pivots. In the case of the
ildings of the Paris Exposition, it was estimated that a range of
100° F. would produce a change in level of 2⅞ in at the center
CHED RIBS are usually built of plates, angles, or channels, with

riveted connections and frequently with a solid-plate web at the bottom. Th determining of the stresses and detailing of the members and joints require th services of a competent structural engineer; but the illustrations given wi enable the architect to decide on the general shape of the trusses for the purpo of making preliminary drawings and the computations and detail drawings ca be made later.

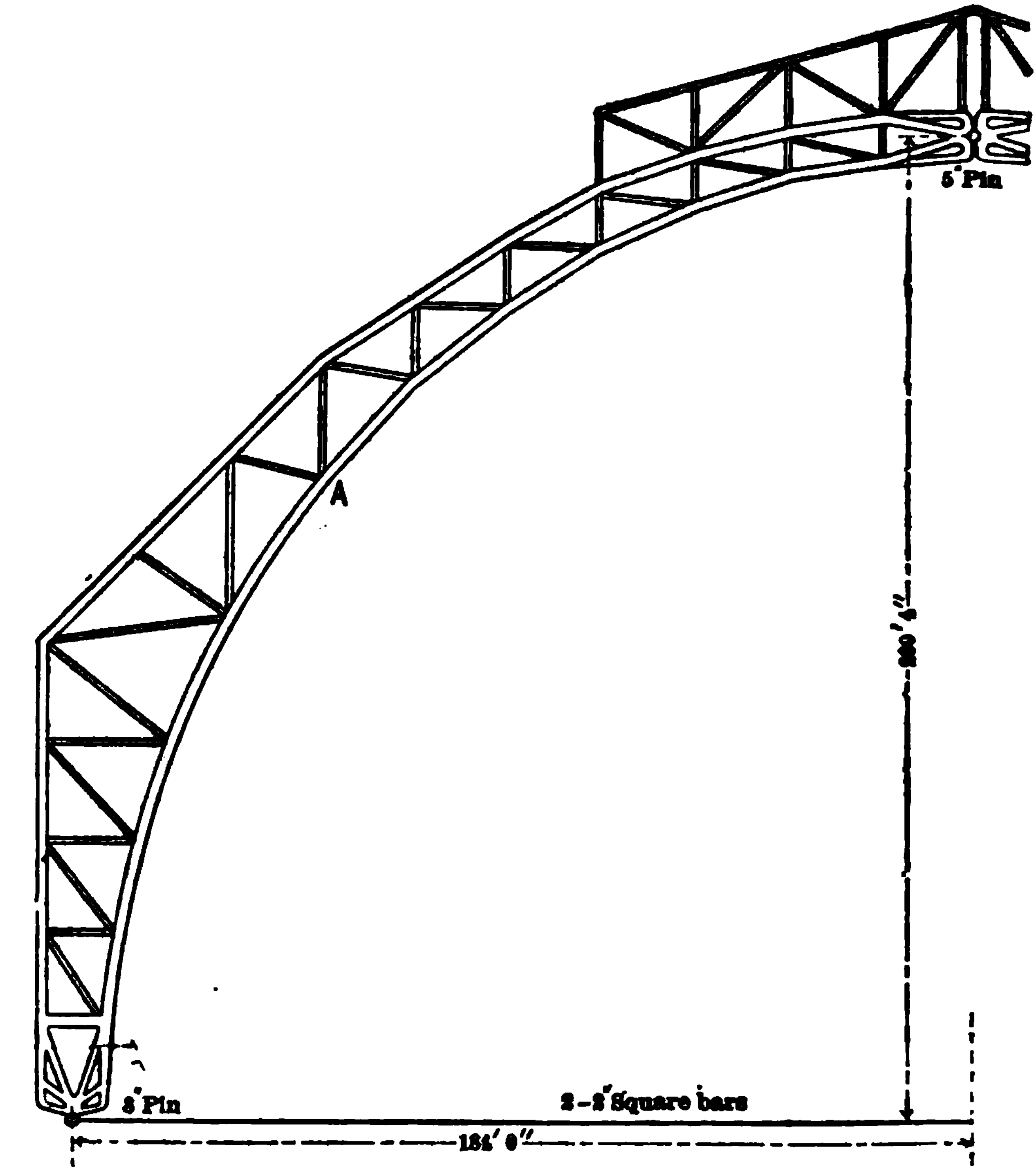


Fig. 72. Half Truss. Three-hinged Arch. Manufactures and Liberal Arts Building Chicago Exposition

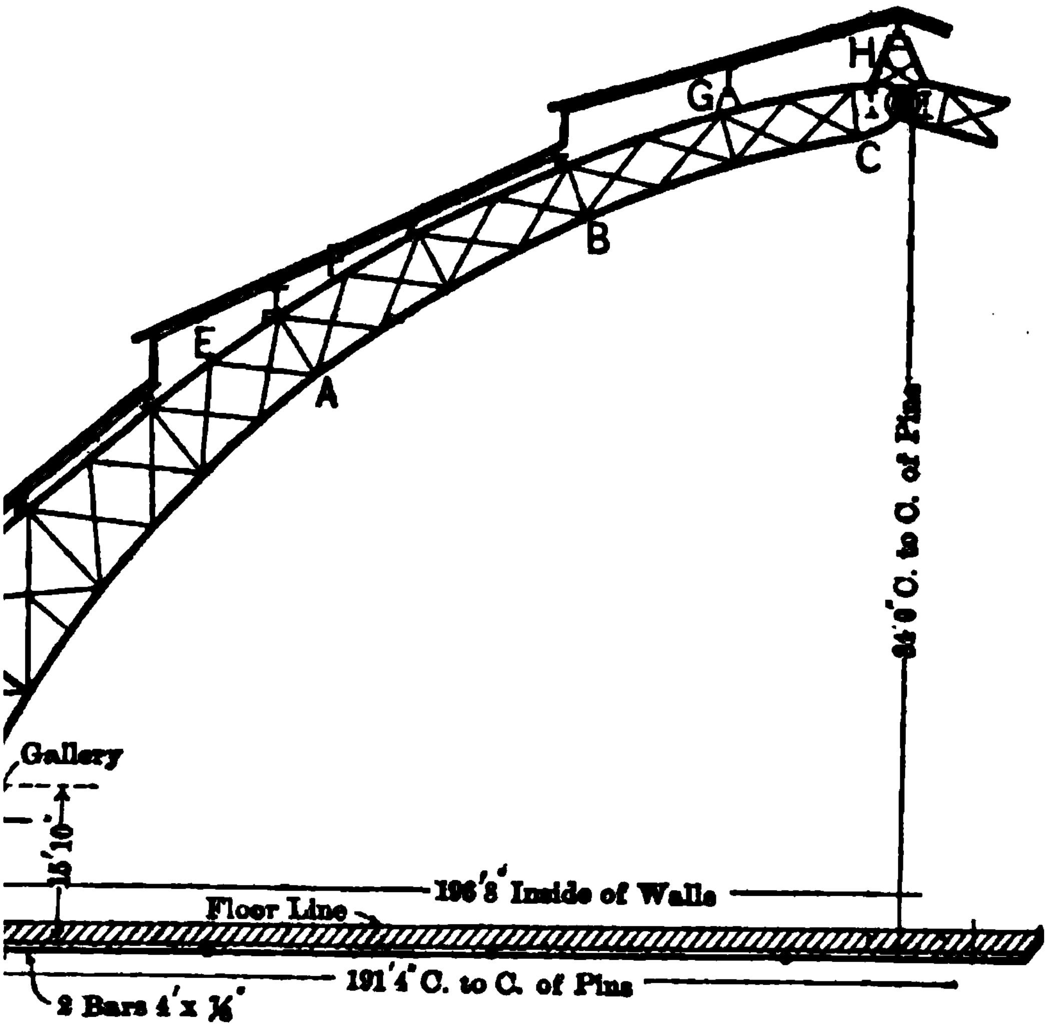
Trusses for Railway Station, Frankfort-on-the-Main, Germany. Th first suggestion for HINGING THE RIBS AT THE CROWN was made by M. Manto a French engineer. The writer believes that the first application of this princip to roof-trusses, at least on a large scale, was made in the train-sheds of the Uni Railway Station completed in the year 1888 at Frankfort-on-the-Main, Germany These trusses have a span of about 184 ft. Engineering Record of Septemb 12, 1891, and March 5, 1892.

Trusses for Machinery Hall, Paris Exposition. The large roof of th Machinery Hall of the Paris Exposition of 1899 was supported by trusses o

; span being 368 ft and exceeding anything hitherto attempted in Since then trusses of this kind have been frequently used for exhibition-halls, train-sheds, armories, and similar buildings.

›r Manufactures and Liberal Arts Building, Chicago Exposi- 2 shows the half-truss of one of the THREE-HINGED ARCHES sup- ›of of the Manufactures and Liberal Arts Building of the Chicago Engineering News, September 1, 1892.

›r Drill-Hall, Brooklyn, N. Y. Fig. 73, in a similar manner, lf-truss of one of the THREE-HINGED ARCHES over the drill-hall of



. Half Truss, Three-hinged Arch, Drill-hall, Brooklyn, N. Y.

ıent Armory, Brooklyn, N. Y. Engineering Record, December scription of the arch shown in Fig. 74 is given in the Engineering :mber 19 and December 24, 1892. The horizontal thrust due to ; small.

Arches. When there are only two pins, usually at the sup- :s become TWO-HINGED ARCHES. As in the case of three-hinged ay be a tie or the supports may be entirely depended upon to ›ntal thrust.

Live-Stock Pavilion, Chicago, Ill. In the Engineering News 5, there is a description of the TWO-HINGED ARCHES supporting building. The arch span is 198 ft, the rise 54 ft and the truss- Each truss has a tie consisting of one $2\frac{9}{16}$-in round rod.

Railway Station, Cologne, Germany. This station, owned Railways, has TWO-HINGED ARCHES supporting the roof of the : arch-span is 209 ft 6 in and the rise 79 ft. There is a brief . the Engineering News, October 6, 1892. A number of roofs

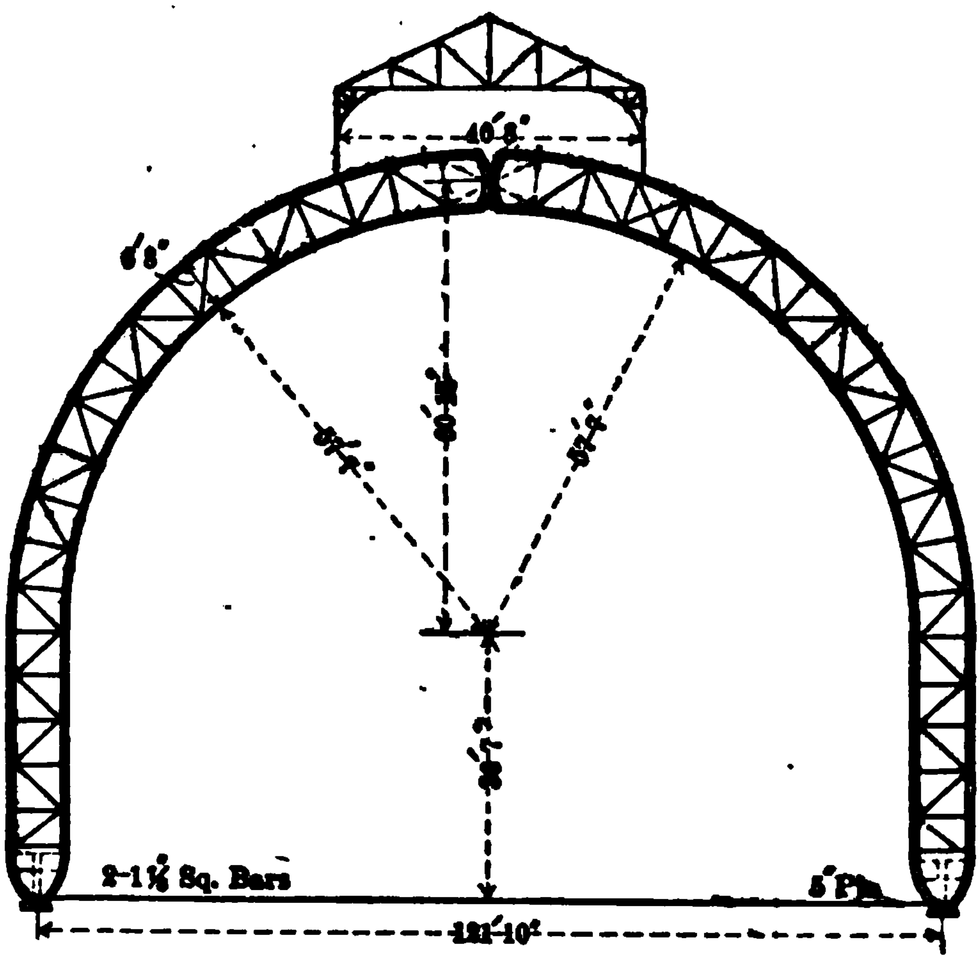


Fig. 74. Three-hinged Arch, Machinery Hall, Chicago Exposition

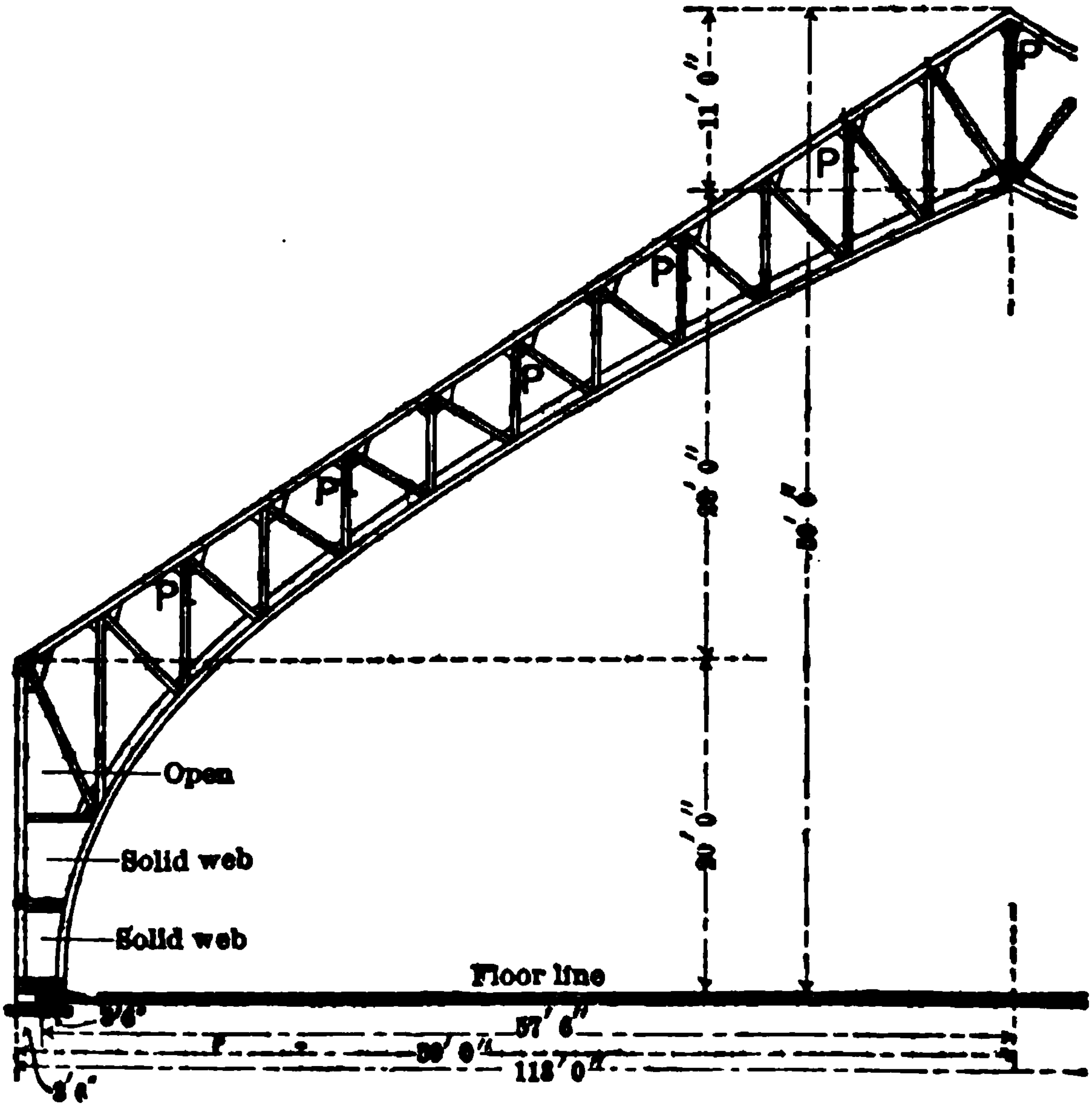


Fig. 75. Two-hinged Arch, Exposition Hall, Providence, R. I.

d by structures similar to that shown in Fig. 75. While such a frame r a TWO-HINGED ARCH, owing to the lack of freedom at the supports, ever, for all practical purposes, be so considered.

List of Buildings with Trusses of the Two-Hinged-Arch Type

Name	Span	Spacing
	ft	ft
ory, Pawtucket, R. I.	82	24
ery, Portland, Me.	92	25
aix Hall, Brockton, Mass.	96	24
ory, Northampton, Mass.	100	24
e Rink, Hartford, Conn.	104	25
sition Hall, Providence, R. I.	118	24.5
ory, Cleveland, Ohio	120	23 to 25
ory, Boston, Mass.	122	30
ory, 22d Reg., New York City	176	24.5
ry, Brooklyn, N. Y.	196	35

ures are described in Building Construction and Superintendence, Part idder and are similar to the type shown in Fig. 75.

es, or arches without hinges, are seldom employed in buildings. examples cited above the structures have the appearance of being ds, but a closer inspection indicates that they are not sufficiently arrant their being classed as FIXED ARCHES.

5. Cantilever Trusses

inciples. A CANTILEVER BEAM or CANTILEVER TRUSS is that ger beam or truss which extends beyond one of the supports, as 79 and *A* in Fig. 80. The overhanging portion *B* is called the M and the portion *C* the ANCHOR-SPAN. The cantilever-arm may nd another beam or truss. The term CANTILEVER was originally te a projecting beam which served as a bracket; in engineering note a beam or girder fixed at one end, by being either built into more commonly the case, extended a sufficient distance beyond rm an anchorage. Thus in Fig. 76, which shows a beam resting , *B* is the cantilever or cantilever-arm and *C* the anchor-span or s obvious that if this entire beam were uniformly loaded the l carry the greater part of the total load; and also, that an *V*, at the end of the cantilever, might cause a negative reaction t the support *D*, in which case the reaction at *P* would exceed beam, unless the negative reaction at *D* is considered as an Although both conditions of loading occur in practice, the the truss usually requires an anchorage rather than a support . As applied to roof-construction some such arrangement as 77 is generally required to make this method of support prac- wide middle span, with shorter spans or aisles on each side of ver-arm is usually made from ¼ to ⅓ the middle span and a RUSS, represented by *S*, supported by the arms of the canti- support the rest of the roof. In all such cases, therefore, can- st be used in pairs, one on each side of the building; and there passages outside of the principal span to permit the use of the

outer or anchor-spans. This arrangement is generally found in auditoriu armories, exhibition-halls and similar buildings, and is sometimes convenient adapted also to other classes of structures. Of course, in a large building a bea consisting of a single member such as is shown in Fig. 77 could not be use

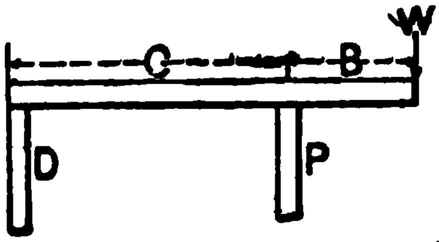


Fig. 76

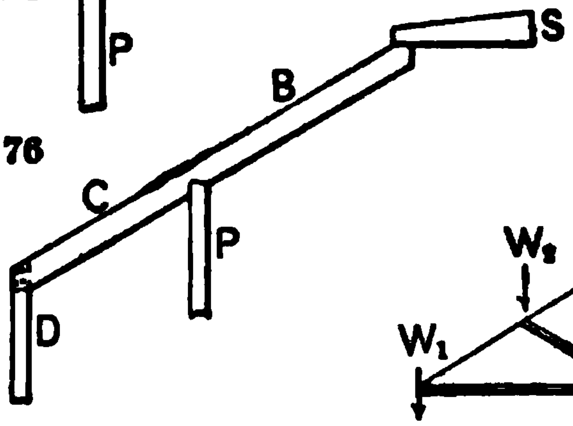


Fig. 77

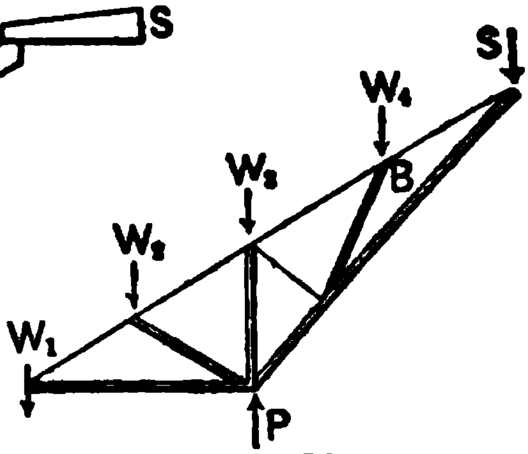


Fig. 78

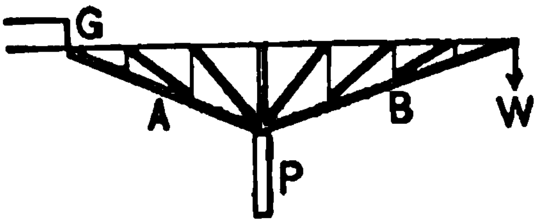


Fig. 79

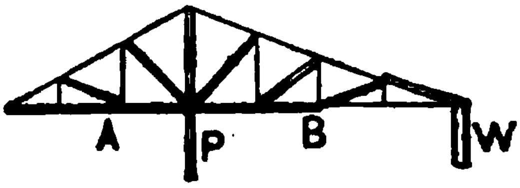


Fig. 80

Figs. 76 to 80. Cantilevers and Cantilever Trusses

but the principle of construction is the same whether the cantilever is a si member or a large truss. Fig. 78 is the diagram of a truss which takes the p of the beam *CB* in Fig. 77, the single lines representing the tension-memb and the double lines the compression-members. Fig. 81 shows the comp arrangement of two of these trusses with the accompanying middle truss, fo

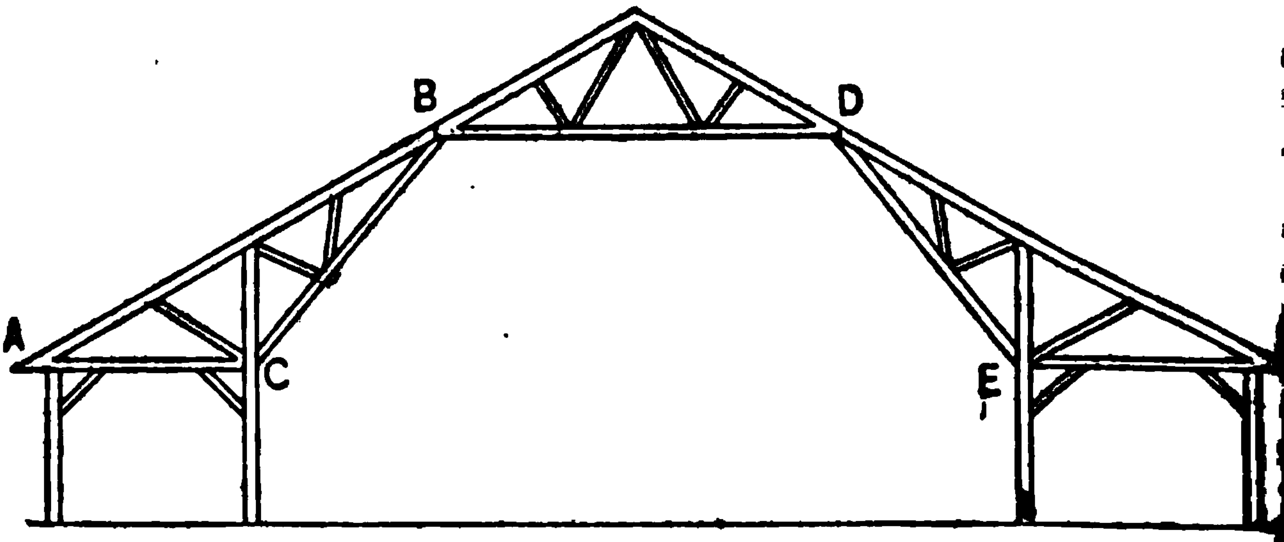


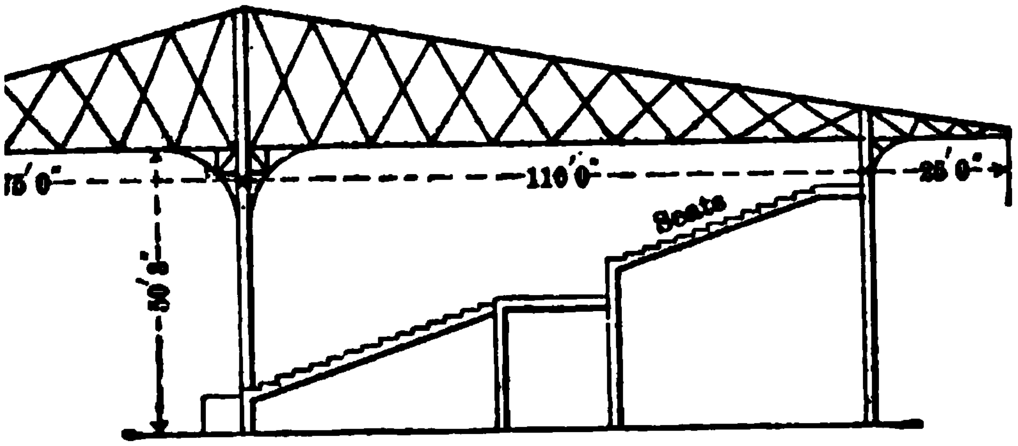
Fig. 81. Suggestion for Wooden Cantilever Truss

entire roof. The truss-principle shown in these figures may be develop almost any extent. The lower chord may be curved, but the general of the truss is best adapted to those roofs in which a wide middle part be supported by cantilevers. For bridge-trusses or floors the form sho Fig. 79 may be used; while for shed and platform-roofs, open on one side, t

own in Fig. 80 are about the only ones practicable. In this latter ortions of the arms are such that only a slight support is required onsequent compressive stress developed in the lower portion of the

s and Disadvantages of the Cantilever Truss. The cantilever some special advantages. The clear height in the middle is greater btained with any other type excepting the three-hinged arch; its light and graceful, and there is no horizontal thrust and consecessity for tie-rods. The particular advantage of this truss for very that it can be erected without scaffolding under the middle part, work this is considered as its only advantage. It is claimed by it engineers that the CANTILEVER TYPE OF TRUSS is not an economiot as desirable for spans of 150 ft or more as the THREE-HINGED s not as readily lend itself to methods of allowing for expansion n as the THREE-HINGED ARCH, the BOWSTRING TRUSS, or the QUADuss. For certain classes of buildings, however, and especially ldle span does not exceed 150 ft, it can perhaps be used with tural effect than is possible with other types, the cost remaining ie. For roofing platforms, grand-stands, etc., where an outer desired, it is the only type available.

rand-Stand, Monmouth Park, N. J. Fig. 82 is a diagram of TILEVER TRUSSES supporting the roof of the grand-stand at this



2. Cantilever Truss, Grand-stand, Monmouth Park, N. J.

details of which were published in Architecture and Building, 90. This is an instance in which the cantilever was the only could be used and the form adopted is both simple and economiseen from the drawing, the main supporting column extends to uss, as is usually the case with cantilever trusses, and the truss side of it. The upper and lower chords are made of two angles The bracing consists of angle-bars used in pairs and varying ¼ in to 3 by 3 by 5/16 in, the whole frame being connected by

the Fore River Ship-building Shed, Quincy, Mass. In Record, July 26, 1902, there is a description of the roof of this h the CANTILEVER TRUSSES have an overhang of 60 ft.

for Grand-Stand, Empire City Trotting Association, These trusses have CANTILEVER-ARMS at each end, 25 ft 6 in 5 ft 6 in on the other. The intermediate truss has a span of cture is described in the Engineering Record, February 10, mples of CANTILEVER ROOFS are given in Building Construction nce, Part III, by F. E. Kidder.

CHAPTER XXVII

STRESSES IN ROOF-TRUSSES

BY

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1. Roof-Loads. Data, Weights, Materials, Methods

Data for Roof-Trusses. Before the stresses in a roof-truss can be det mined it is necessary to decide upon the character of the roof-covering, t method of supporting it between the trusses, the geometrical shape and sp of the trusses and the spacing of the trusses.

Roofing Materials for Pitched Roofs. The materials suitable for coveri pitched roofs are slate, burnt-clay tiles, metal tiles or shingles, wooden shingl corrugated iron, tin with standing seams, standing-seam steel roofing a various kinds of ready roofing. The least slope to which these materials m be laid without danger of leaks, the weight per square foot of roof and the co parative cost are indicated in Table I. The cost, however, can only be conside as approximate, as it varies for different materials, localities and the scales wages.

Table I. Covering Materials for Pitched Roofs

Material	Least rise of rafter in 12 in	Comparativ cost per square
Slates, black	8	$7.00 to $12.(
Slates, green	8	7.00 to 10.(
Slates, red	8	12.00 to 17.(
Burnt-clay tiles, interlocking pattern	7	15.00 to 25.(
Tin shingles, painted	6	8.00 to 10.(
Galvanized-iron tile, painted	6	13.00 to 15.(
Cedar shingles, stained or painted	6	3.80 to 7.(
Corrugated iron, painted	3	4.00 to 4.(
Standing-seam steel roofing, painted	2	4.00 to 4.(
Ready roofing	1	3.50 to 4.(

Roofing Materials for Flat Roofs. Flat roofs or roofs having a fall from ½ to ¾ in to the foot are usually covered with tar and gravel, asph ready roofing, or tin with lock-and-solder joints. A good tin roof costs ab $8.00 a square, not including the painting. The other kinds vary from $ to $4.50 a square.

Manner of Supporting the Roof from the Trusses. Wooden roofs, s ported by wooden trusses, require common or jack-rafters to support the shea ing or slate, and generally purlins to support the rafters, although in some cas may be more economical to span the rafters from truss to truss (Fig. 17, Chap XXVI). When slates or burnt-clay tiles are used on steel roofs, they are usu secured to steel angles, running parallel with the walls and spaced from 8 10½ in apart, as may be necessary to accommodate the size of the slates

an is not more than 6 or 7 ft, the angles may be fastened to the As a rule, however, when slates or tiles are to be used, it is cheaper sses from 16 to 20 ft apart, and to use purlins and jack-rafters to aller angles. Quite often, wooden rafters and sheathing are trusses. This is more economical, but of course increases the otected steel is little if any better than wood. If corrugated d for roofing, the most economical construction for steel roofs is sses from 16 to 20 ft apart, and to use light I beams for purlins, ¼ ft on centers, as in Fig. 52, Chapter XXVI, the corrugated ed to the purlins by straps. If warm air comes in contact with a corrugated roof, either the roofing should be laid on boards, anticondensation lining should be provided, as otherwise the air will condense and fall on the floor or objects below. Flat quire rafters and sheathing, or fire-proof filling between the

usses. From the above it is seen that the economical spacing pends to a great extent upon the kind of roofing that is used, he span. As a general rule, however, the most economical as follows:

RUSSES under 80-ft span, from 12 to 16 ft on centers.
RUSSES over 80-ft span, from 16 to 24 ft on centers.
SSES under 80-ft span, from 16 to 20 ft on centers.
SSES over 80-ft span, from 20 to 40 ft on centers.

f a number of steel trusses of wide span is given in Chap- n the distance between the trusses exceeds 16 ft for wooden steel roofs, it is generally necessary to use trussed purlins. pon the kind of truss to be used, the spacing of the trusses and ion, a section-drawing of the roof should be made, showing e truss, the points at which the purlins are to be supported, porting the ceiling, if there is one, and any other loads that ed by the trusses. The section and truss-drawing, with the hts of roofing-materials, will furnish the necessary data for ds at each joint. Until the stresses have been determined, embers computed, and the joints detailed, an exact drawing t, of course, be made; but in order to compute the loads and ssary to know the positions of the joints, and these can be ficient accuracy before the exact sizes of the members are pter XXVI gives sufficient information regarding the various enable one to decide upon the height and the number and struts and ties; and the sizes of the members can be approx- minary drawings.

ng-Area Supported at Any Joint. Calculations for the re always based on the assumption that the loads are trans- , and that the members are free to move at the joints as if e actual joints may be made with riveted or other connec- t the joints are, of course, equal to the reactions of the pur- ams or principals, if these receive the ceiling-joists or rafters. he roof or ceiling is uniformly distributed, as is usually the ethod of computing the joint-loads is to determine the roof butory to the joint, and to multiply this area by the weight or . The area contributory to any joint is equal to the product sured half-way to the next joint, on each side, by the dis- -way to the next truss or wall. on each side. Thus if Fig. 1

represents truss 1, of Fig. 2, the roof-area contributory to joint 2 is, in square fe
$\frac{8+14}{2} \times a$. For truss 2, the area supported by the same joint is $\frac{14+12}{2} \times$
or, if we let D represent the length in feet of roof or ceiling supported at e
joint, the area in square feet supported by joint 2 is $a \times D$, and the area s

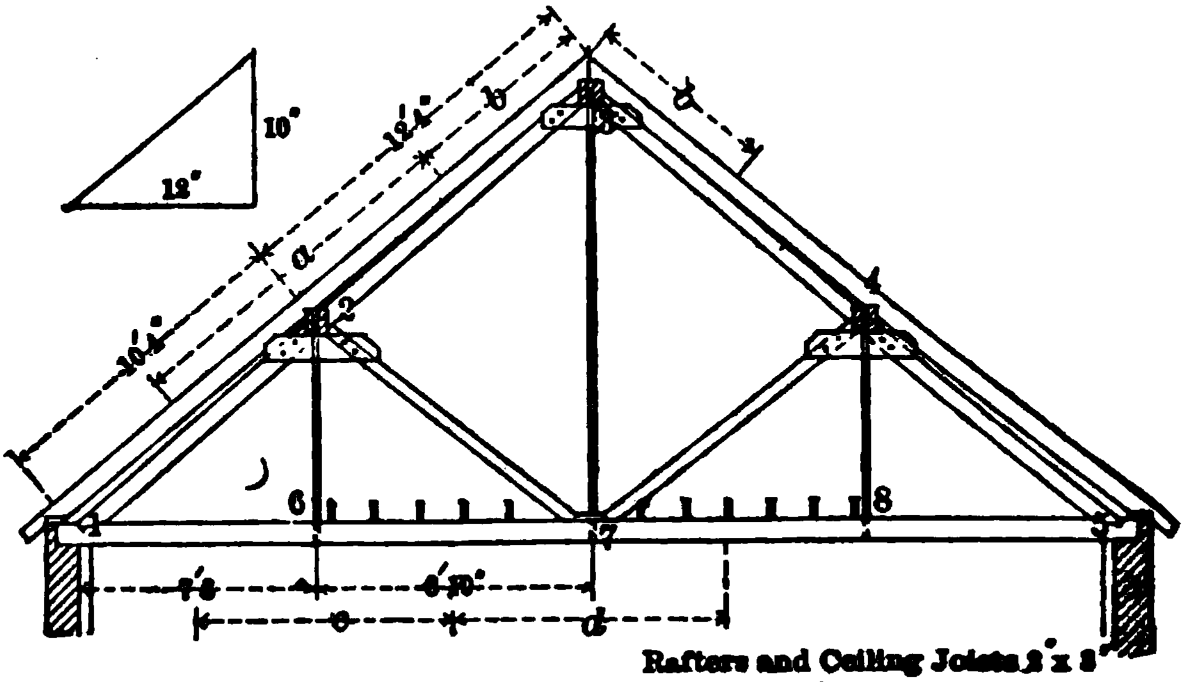


Fig. 1. King-rod Truss

ported by joint 3 is $2b \times D$. In the same way, the ceiling-area supporte
joint 6 is $c \times D$, the arrow-heads being half-way between the joints. It m
no material difference in the joint-loads whether the common rafters are
ported on purlins or whether they rest on the top chord of the truss, provi
the purlins come at or close to the jo
and the load is uniformly distribu
Thus the width of the ceiling cont
utory to joint 7 (Fig. 3) is equal t
just the same as in Fig. 1. The arra
ment in Fig. 1 produces cross-bea
stresses in the tie-beam, while tha
Fig. 3 does not. When the trusses
spaced a uniform distance apart, D,
2, is, of course, equal to the dist
between centers of trusses. When
trusses are not spaced uniformly,
equal to one-half the distance from
center of the truss on the left to
center of the truss on the right. Wh
purlin is more than 12 in from a joi
the roof-area is not symmetrical,
often the case at hips and valleys,
joint-load is determined by the pri
of the REACTION OF BEAMS, as explai

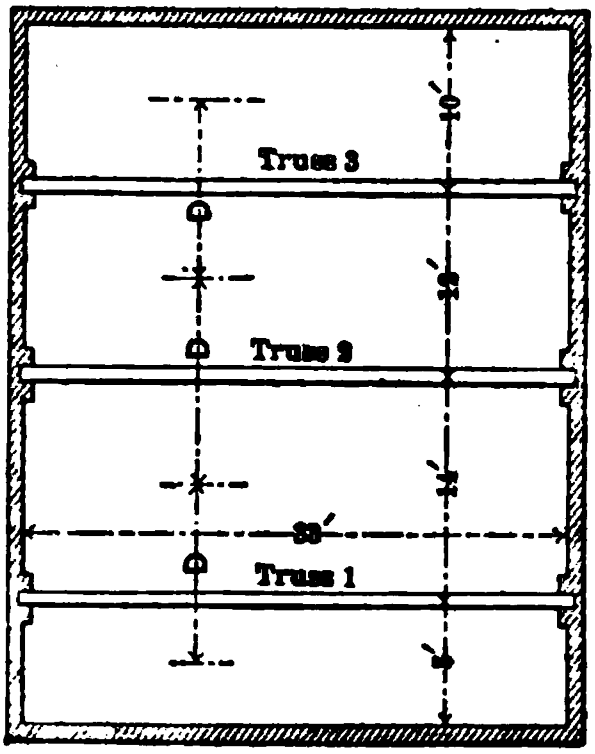


Fig. 2. Plan of Wall and Trusses

Chapter IX. Examples showing the computation of joint-loads are giv
little farther on.

Roof-Load per Square Foot. By the term ROOF-LOAD is meant the weig
the materials composing the roof, trusses and purlins, an ample allowanc

an allowance for wind-pressure. The weight of the materials
EAD LOAD. Snow is generally considered a LIVE LOAD, acting
e pressure due to the wind is always assumed to act normal to,
les to, the surface of the roof; but for trusses of less than 100-ft

mes. (See, also, Figs. 12, 53 and 54 and Chapter XXVIII, Fig. 1)

r combined with the DEAD LOAD and SNOW-LOAD and treated
oad. This does not apply to the Fink and fan types. (See

puting Dead Loads. The DEAD LOAD of any roof may be
fficient accuracy from the following data:

e II. Weights per Square Foot of Roof-Surface

t, 2½ lb; 18 in, 3 lb
k, 7¾ lb; ¼ in thick, 9.6 lb (the common thickness is 5/16 in for
r 20 in)
shingles, 11 to 14 lb
style, two parts, 12 lb; new style, one part, 8 lb
style, two parts, 19 lb; new style, one part, 8 lb
f tiles, 11 lb
b
ortar add 10 lb per sq ft
eets, 1½ lb; tiles, 1⅝ lb
or shingles, including one thickness of felt, 1 lb
ainted or galvanized, No. 26, 1 lb; No. 24, 1.3 lb; No. 22, 1.6 lb;
No. 18, 2.6 lb; and No. 16, 3.3 lb
el roofing, 1 lb
ravel roof, 6 lb
ravel roof, 5½ lb
ofing (elaterite, ruberoid, asphalt, etc.), from 0.6 to 1 lb
vanized-iron frame, ¼-in glass, 4½ lb; 5/16-in, 5 lb; ⅜-in, 6 lb
k, 3 lb per sq ft for white pine, spruce, or hemlock; 4 lb for yel-

Table III. Weights of Rafters per Square Foot of Roof-Surface

Size of rafter in inches	Spruce, hemlock, white pine. Spacing in inches, center to center			Hard pine. Spacing in inches, center to center		
	16	20	24	16	20	24
	lb	lb	lb	lb	lb	lb
2×4	1½	1.2	1	2	1.6	1⅓
2×6	2¼	1.8	1½	3	2.4	2
2×7	2⅝	2.1	1¾	3½	2.8	2⅓
2×8	3	2.4	2	4	3.2	2⅔
2×10	3¾	3	2½	5	4	3⅓

Wooden purlins weigh about 2 lb per sq ft of roof-surface when the span is betw
12 and 16 ft.

For steel roofs the sizes and weights of the purlins and rafters should be computed
each particular case.

Weight of Truss. To the weight of the roof-construction proper sho
be added an allowance for the weight of the truss. If trusses could be b
in exact accordance with the theoretical requirements their weight would
directly proportional to the roof-load and span; but as there is always s
extra material, it is impossible to determine the weight of the truss exactly u
it is completely designed. Several tables for the weights of wooden tru
and formulas for steel trusses have been published, but hardly any two of t
are alike. The following are some of the formulas in use:

For Wooden Trusses

$W = 0.04\,L + 0.000167\,L^2$	N. C. Ricker, for trusses like Fi and 5, Chapter XXVI.
$W = 0.50 + 0.075L$	H. S. Jacoby.

For Steel Trusses

$W = 0.75 + 0.075\,L$	Mansfield Merriman and Jacob
$W = 0.6 + 0.06\,L$, for heavy loads; $W = 0.4 + 0.04\,L$, for light loads	C. E. Fowler, for Fink trusses.
$W = \dfrac{P}{45}\left(1 + \dfrac{L}{5\sqrt{A}}\right)$	M. S. Ketchum, for steel mill-b ing trusses.
$W = 0.05\,L + 12/A$	H. G. Tyrrell.

In the above formulas, W = weight of truss in pounds per square foot of h
zontal projection of the roof supported, L = span in feet, A = distance betw
trusses, and P = capacity of truss in pounds per square foot of horizontal p
jection.

Tables IV and V, compiled from a comparison of other tables and form
and from the weights of actual trusses, are sufficiently accurate for the pu
of determining stresses. The weights given are probably slightly in exce
the actual weights of average trusses, as it is preferable to have the error, if
on the safe side. It should be noted that the weights are for each square
of roof-surface, and not for the horizontal area. Table VI gives the a
weights of a number of large steel roofs.

Weights of Wooden Trusses per Square Foot of Roof-Surface*

Span	½ pitch	⅓ pitch	¼ pitch	Flat
	lb	lb	lb	lb
....	3	3½	3¾	4
....	3¼	3¾	4	4½
....	3½	4	4½	4¾
....	3¾	4½	4¾	5¼
....	4¼	5	5½	6
....	5	6	6½	7
....	5¾	6¾	7	8
....	6½	7½	8	9
....	7	8½	9	10

Weights of Steel Trusses per Square Foot of Roof-Surface

an	½ pitch	⅓ pitch	¼ pitch	Flat
	lb	lb	lb	lb
....	5.25	6.3	6.8	7.6
....	5.75	6.6	7.2	8.0
....	6.75	8.0	8.6	9.6
....	7.25	8.5	9.2	10.2
....	7.75	9.0	9.7	10.8
....	8.5	10.0	10.8	12.0
....	9.5	11.0	12.0	13.2
....	10.0	11.6	12.6	14.0

hts and Spacing of Some Steel Roofs of Wide Span, Including , Purlins and Braces, but not Roof-Covering or Rafters

ilding	Type of truss	Span ft	Spacing, center to center of trusses, ft	Weight per sq ft sloping surface, lb	Weight of one truss, tons
ket, R. I...	Fig. 75†	82	24	8.7	6.7
d, Me......	"	92	25	9.7	9
Brockton, ...	"	96	24	8.6	10
thampton, ...	"	100	24	8.0	8.5
Hartford, ...	"	104	25	11.8	11.5
ence, R. I..	"	118	24½	9.5	12.5
Armory..	"	120	23–25	....	....
Mass......	"	122	30	12.4	21
., N. Y...	"	176	24½	....	....
n, N. Y....	"	196	35	....	....

* For scissors trusses, increase one-third.
† Chapter XXVI.

The data for the first seven buildings in Table VI were compiled by H. (Tyrrell, who states that all of the seven roofs were proportioned for sla and plank roofing resting on wide rafters 2 ft apart, supported by steel purli about 10 ft apart. The spans given are measured from center to center of si bearings. Stresses were computed for a dead load of 25 lb per sq ft, a snow-lo of 10 lb per sq ft of sloping surface, and a horizontal wind-load of 40 lb per sq ft a 28-lb-per-sq-ft normal pressure. Data for computing the weights of floors a floor-loads supported by trusses, and for fire-proof construction, may be found Chapters XXI and XXIII.

Snow-Loads. As a basis for making an allowance for snow, Table VII perhaps as good a guide as any that can be given. When snow-guards a placed on a roof, the same allowance is made for a half-pitch as for a one-thi pitch.

Table VII. Allowance for Snow in Pounds per Square Foot of Roof-Surfac

Location	Pitch of roof				
	½	⅓	¼	⅕	⅙ or le
	* †	* †	* †		
Southern states and Pacific slope......	0– 0	0– 5	0– 5	5	5
Central states........................	0– 5	7–10	15–20	22	30
Rocky Mountain states...............	0–10	10–15	20–25	27	35
New England states..................	0–10	10–15	20–25	35	40
Northwest states.....................	0–12	12–18	25–30	37	45

Columns headed by an asterisk (*) are for slate, tile, or metal; those headed b dagger (†) are for shingles.

Wind-Pressure.* For roofs having a pitch of 5 in or more to the foot, allowance must be made for wind-pressure. For trusses of the FINK, FAN, KI or QUEEN TYPES, the usual practice is to include the wind-pressure with t vertical loads, and to make a single allowance for both wind and snow, as duri a gale snow is not likely to stay on a steep roof. When the wind-pressure added to the vertical loads, the allowance for wind and snow combined sho not be less than indicated in Table VIII.

Table VIII. Allowance for Wind and Snow Combined in Pounds per Square Foot of Roof-Surface

Location	Pitch of roof					
	60°	45°	⅓	¼	⅕	⅙
Northwest states............	30	30	25	30	37	45
New England states.........	30	30	25	25	35	40
Rocky Mountain states......	30	30	25	25	27	35
Central states...............	30	30	25	25	22	30
Southern and Pacific states..	30	30	25	25	22	20

No roof-truss should be proportioned for a total load of less than 40 lb per sq of roof-surface except flat roofs in warm climates. For trusses having sp exceeding 100 ft (except trusses for flat roofs) and for trusses in which a part

* (See, also, Chapter XXX, page 1199, and pages 1394 and 1717.)

duce maximum stresses, or call for COUNTERBRACING, as is the case ERAL TRUSSES, and trusses with CURVED CHORDS, the stresses for all oadings should be found separately and each member of the truss for the maximum stress to which it may be subject under any ination of loads. For determining the stresses due to wind-pres- force of the wind is usually assumed to act in a direction normal, t-angles, to the slope of the roof. This force is commonly based on ind, producing a pressure of 30 lb against a vertical surface. This a wind-velocity of nearly 100 miles per hour. According to ıula,

$$P = 0.0032\ V^2$$

pressure in lb per sq ft against a surface normal to the direction ıd V = the velocity in miles per hour. For $P = 30$ lb, $V = 96.3$ ormal pressure per square foot of roof-surface corresponding to and 30 lb per sq ft against a vertical surface is given in Table IX.

. **Wind-Loads in Pounds per Square Foot of Roof-Surface***

Inclination of roof	Normal pressure P_n, pounds per square foot	
	$P = 30$ lb	$P = 20$ lb
....	5.1	3.5
....	10.1	6.8
....	14.6	9.6
h..	19.8	13.1
h..	22.4	14.0
....	24.0	16.0
h..	25.5	17.0
....	26.7	18.2
....	28.3	18.9
....	30.0	20.0

ı Table IX are based on Duchemin's formula,

$$P_n = P \frac{2 \sin\theta}{1 + \sin^2\theta}$$

e pressure per square foot on a vertical surface, P_n the normal essure and θ the angle of inclination of the roof with the hori- ıd not only produces a pressure upon the windward side of the n upon the leeward side; therefore all roof-covering should be l, all joints in the trusses so constructed that they will resist pression, and the trusses themselves securely anchored to the

Loading for which Stresses should be Found. To deter- ım stresses under any possible condition of loading, stresses or the following cases:

due to permanent DEAD LOADS,
vering only one side of roof,
vering entire roof,
side of truss nearer the expansion-end,
side of truss nearer the fixed end.

lso, Chapter XXX, page 1199, and pages 1394 and 1717.)

It is generally assumed that the maximum wind-pressure and the snow-lo cannot act on the same half of the truss at the same time; hence the combi tions for maximum stress will be either cases 1 and 3 or cases 1, 2, and 4 or If the trusses are supported on iron columns instead of on walls the wind-fo is transferred to the foundations through the columns, producing a bendi moment in the columns. The stresses in the columns, trusses and knee-bra should therefore be determined for the wind-pressures against the side of t building and roof. These pressures are obtained by multiplying the area the vertical surfaces by the full pressure per square foot and the area of t roof by the normal component, given in Table IX.

Kansas City Auditorium. For the trusses supporting the roof of the Ka sas City Auditorium (Fig. 66, Chapter XXVI) stresses were computed for t following conditions: First, full dead and live load on both galleries and t roof-garden, and wind-pressure due to a velocity of 45 miles an hour; seco full dead load, snow-load, and gallery live load, wind-pressure 10 lb and load on roof-garden floor; third, full dead load and 50 lb wind-pressure; fourt full dead load and wind-pressure at 45 miles an hour, and full live loads gallery and roof-garden on one side only. Snow-loads throughout were tak at one-third of the dead load. Examples showing manner of combining t stresses due to different conditions of loading are given on pages 1114 and 1123

2. Examples of the Computation of Roof-Loads*

King-Rod Truss. Example 1. The first example considers the roof a truss shown in Fig. 1, page 1048, which it is assumed represents truss 2 of Fig. It is assumed that the timber is to be common white pine and that the roo to be covered with 3/16-in slate of medium size on 7/8-in sheathing. The ceil is to consist of lath and plaster. The dead load of roof and truss per squ foot of roof-surface is made up as follows:

	lb per sq
For slate	7¼
For sheathing	3
For rafters	3
For purlins	2
For truss	3
Total	18¼

For wind and snow-load combined there should be allowed about 28 lb (pitch being about 40°), which makes a total roof-load of 46¼ lb. To av fractions, however, the load is assumed to be 48 lb per sq ft. As the dista to truss 1, Fig. 2, is 14 ft and to truss 3, 12 ft, the length of roof supported the truss is 13 ft. The roof-area supported by the purlins at joint 2 is equa the distance *a* multiplied by 13 ft; and *a* is one-half the distance from the w plate to the ridge-purlin, or 22 ft 8 in divided by 2, or 11 ft 4 in, or 11⅓ Hence the roof-area supported at joint 2 is 11⅓ by 13 ft, or 147⅓ sq ft. roof-area supported by the purlins at joint 3 is 2 *b* by 13 ft, or 12⅓ by 13 ft 160⅓ sq ft. Multiplying the roof-areas by the load per square foot, 48 lb, t results 7 072 lb for the load at joint 2; and 7 696 lb for the load at joint 3. load at joint 4 is equal to that at 2, as the truss is symmetrical. The ceil loads at joints 6 and 7 are computed next. The ceiling-area supported at j 6 is $c \times 13$ ft, or 8¼ by 13 ft, or 107¼ sq ft. The area supported at joint 7 is by 13 ft, or 114⅝ sq ft. The actual weight of the ceiling per square foo

* In the following five examples all loads are considered as acting vertically.

ists and 10 lb for the lath and plaster; but where there is a large ible to be used for storage it is well to make a small allowance, sq ft, for any extra attic-load. Therefore, 18 lb per sq ft is he weight of the ceiling, which makes the weight at joints 6 and by 18 lb per sq ft, or 1 930 lb; and the weight at joint 7, 114⅚ sq r sq ft, or 2 067 lb. As soon as computed, the roof and ceiling-e marked on a truss-diagram, as in Fig. 10. The roof and ceiling-ı are transmitted directly to the wall and need not be taken into ermining the stresses in the truss.

182. Example 2. It is required to compute the joint-loads for n in Fig. 3, page 1049. All timber is to be of spruce and the roof d with shingles on 1-in sheathing. The ceiling is to be of lath The dead load is:

	lb per sq ft
shingles	2½
sheathing	3
rafters	2¼
purlins	2
truss	3
dead load per sq ft	12¾
for wind and snow	30
roof-load in pounds per square foot	42¾

of the ceiling it is well, for a truss of this kind, to allow at least It will be assumed that the trusses are to be spaced uniformly . Then the roof-area supported at joint 2 is 9⅚ by 15 ft, or the load at this joint is 6 306 lb. The purlin at joint 3 supports point midway to joint 2, to the ridge, or $b = 4$ ft 11 in + 8 ft 5 in, he roof-area supported at this joint is 13⅓ by 15 ft, or 200 sq is 8 550 lb. The loads at joints 4 and 5 are equal respectively 2. For the ceiling-loads at joints 7 and 8 there is an area to be to 12⅙ by 15 ft, or 182½ sq ft, which, multiplied by 20, gives

183. Example 3. For this example the church-roof shown in is considered. In this roof the trusses take the place of the g-beams, the sheathing spanning from truss to truss and the ıg being nailed to 1¼ by 2½-in furring strips, spaced 12 or 16 in ming that the parts of the trusses have the dimensions indi-e, and that the wood is white pine, the actual weight of one 00 lb. The roof-area supported by one truss is 170 sq ft, and of the trusses is about 7 lb per sq ft of roof-surface. This an twice that given in Table IV, owing principally to the close sses and also to the small dimensions of their members. The thing and shingles is about 5½ lb and 30 lb is allowed for wind-of is too steep for snow to lodge on it. This gives a total roof-r sq ft of sloping surface. For the weight of the ceiling 12 lb as no load other than its own weight is likely to come upon it. ported at joint 2 is 10⅚ by 2½ ft, or 27 sq ft. The area sup-and 5 is equal to 12⅓ by 2½ ft, or 31 sq ft for each. The ted at joint 3 is 14⅓ by 2½ ft, or 35½ sq ft. Multiplying the corresponding loads per square foot, there results 1 148 lb

for the load at joint 2, 1 318 lb for each load at joints 4 and 5, and 426 lb the load at joint 3.

Truss over Car-Barn. Example 4. In this example the roof is of co gated iron, supported by a steel truss of the shape shown in Fig. 55, Cha XXVI. This truss supports nothing but the corrugated iron, the purlins the pressure due to wind and snow, the use of the building not requiring suspending of any load from the trusses. In figuring the dead loads for s a roof, the sizes of the purlins and the gauge of the iron should first be defini

Fig. 4. Scissors Truss. (See, also, Fig. 24 and Chapter XXVIII, Fig. 2)

fixed, so that the weight per square foot of roof may be accurately determ In this instance the purlins are 5-in I beams spaced 4 ft 9 in on centers weighing 10 lb per linear foot. The weight of the purlins per square fo roof is therefore equal to 10 lb divided by 4¾, or 2.1 lb. For a span of 4 f the corrugated iron should be No. 18 gauge (see Corrugated Iron, Part III, 1601) weighing 2 lb per sq ft. For the weight of the truss and bracing the v taken is that given in Table V for a span of 100 ft and ¼-pitch, 10.8 lb.* gives a total dead load of 14.9 lb per sq ft of sloping surface.

For wind and snow we should allow 22 lb per sq ft if the building is sit in the Central states, making the total roof-load 36.9 lb per sq ft. It is generally recommended, however, that no roof should be designed for a less, all told, than 40 lb per sq ft; the joint-loads, therefore, should be com on that basis. The only loaded joints in this truss are those under the pu The trusses are spaced 19 ft 2¼ in and the purlins 4 ft 9 in on centers, th area supported at each upper joint being 91 sq ft. The joint-loads, the should be figured at 3 640 lb. Even for the locality in which it was buil

* The actual weight of this truss and bracing was 4 lb per sq ft of sloping which is remarkably small.

: roof; and it would hardly be considered safe for states further

Flat Roof. Example 5. This truss is for a flat roof (Fig. 5). of spruce and there is a five-ply gravel roof and a plastered ceiling. oad we have,

	lb per sq ft
roofing	6
sheathing	3
rafters	2¼
purlins	2
truss, about	4¼
dead load in pounds per square foot	17½

s required for wind-pressure, but the snow-load is a large per- otal load in any of the Northern states, as indicated in Table VII.

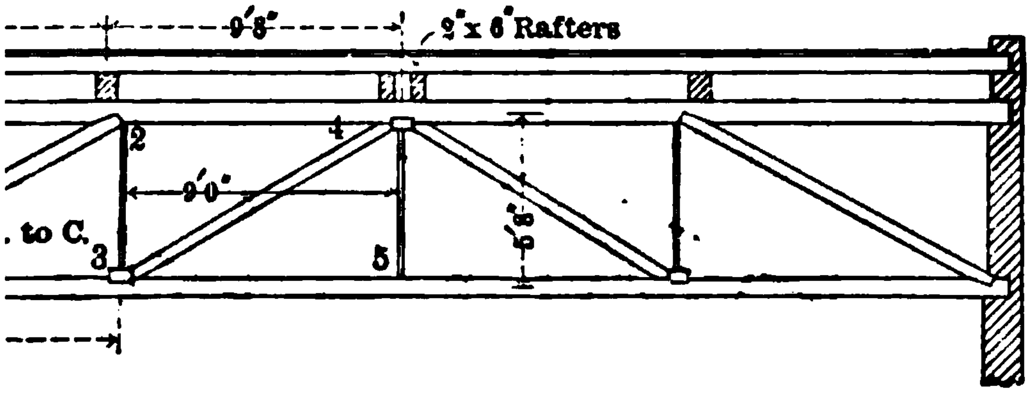


Fig. 5. Howe Truss

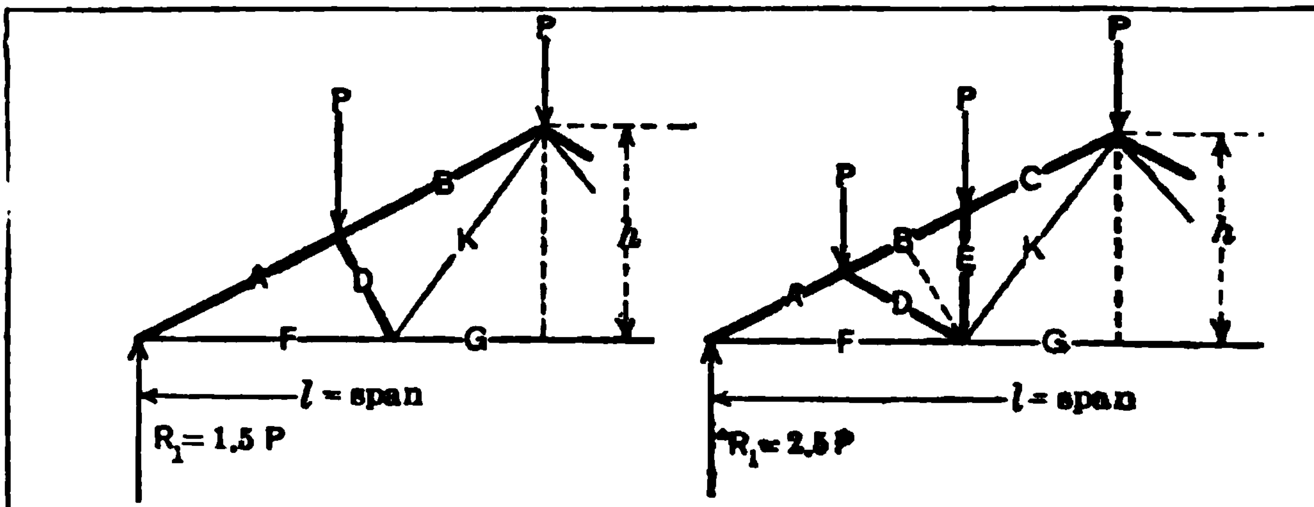
ie building is located in one of the Central states, 30 lb per sq ved for snow, making the total roof-load 47½ lb. The plaster eiling-joists weigh about 12¼ lb and as the roof-space is not for storage, 13 lb per sq ft is a sufficient allowance for the ceil- hat the trusses are to be uniformly spaced, 14 ft on centers, orted at joint 2 is 9½ by 14 ft, or 133 sq ft, and the area sup- 9⅝ by 14 ft, or 135½ sq ft. The ceiling-area supported at ft, or 130⅝ sq ft and at joint 5, 9 by 14 ft, or 126 sq ft. Mul- iese areas by the corresponding load per square foot, we have d at joint 2, 6 428 lb at joint 4, 1 699 lb at joint 3, and 1 638 lb actice it is hardly worth while to compute the stresses closer it the loads may as well be put down at an even 50 or 100 lb obtained by computation. When the roof is supported by often some joints of the truss which have no load. Thus for Fig. 16, Chapter XXVI, there are no loads on joints 2, 6 and supported at joint 4 (Fig. 16) is equal to one-half the distance the distance halfway to the truss on each side. If the lower ing-joists, there is a load at each of the joints 3, 5, 7, 9, etc. can be drawn for any arrangement of loads, the important oute the loads exactly as they are placed on the truss.
oles illustrate fairly well the method of computing the loads of trusses. Other special cases of loading should be com- principles.

3. Determination of Stresses by Computation

Stresses. To determine the stresses, a DIAGRAM OF THE TRUSS, compos of single lines representing the central axial or median lines of the truss-membe should first be carefully drawn to a scale and the loads at the different joi indicated by arrows and numbers as in Figs. 10 and 12. If the center lines the members, as they are actually placed, do not intersect at common poin they must be made to do so in the diagram, as the stresses can be COMPUTED o on the assumption that the center lines of all members meeting at any jo intersect at a common point. In wooden trusses it is not always practicable place the members so that their center lines meet in a common point at ea joint; but this condition should obtain as nearly as practicable, and in st trusses the joint-connections should be made so that the lines passing throu the centers of gravity of the cross-sections of the members meeting at a joi intersect in the same point.

Table X. Coefficients for Determining the Stresses in Simple Fink and Fan Trusses

WHEN PANEL-LOADS ARE ALL EQUAL



Simple Fink Truss Simple Fan Truss

To find the stress in any member, multiply its factor by the panel-load, P

SIMPLE FINK TRUSS

Member	Kind of stress	$l/h=3$	$l/h=3.464$ $=30°$	$l/h=4$	$l/h=5$
A	Compression	2.70	3.00	3.35	4.04
B	"	2.15	2.50	2.91	3.67
D	"	0.83	0.87	0.89	0.93
F	Tension	2.25	2.60	3.00	3.75
G	"	1.50	1.73	2.00	2.50
K	"	0.75	0.87	1.00	1.25

SIMPLE FAN TRUSS

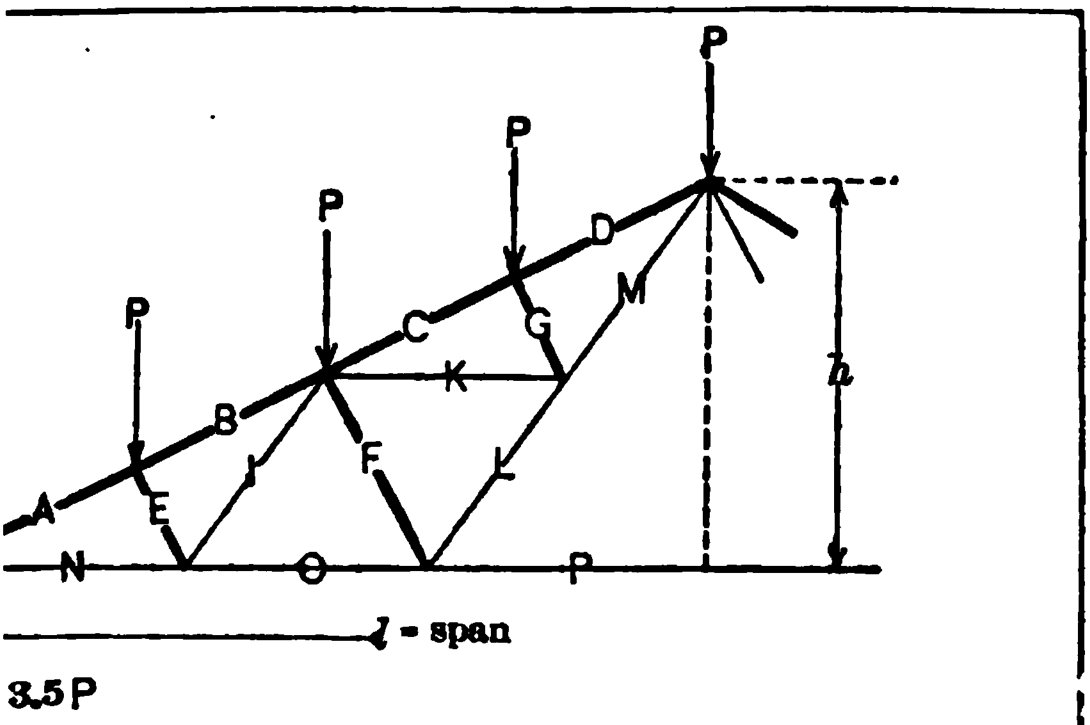
Member	Kind of stress	$l/h=3$	$l/h=3.464$ $=30°$	$l/h=4$	$l/h=5$
A	Compression	4.51	5.00	5.59	6.73
B	"	3.54	4.00	4.55	5.59
C	"	3.40	4.00	4.70	5.99
D	"	0.93	1.00	1.08	1.21
E	"	0.93	1.00	1.08	1.21
F	Tension	3.75	4.33	5.00	6.25
G	"	2.25	2.60	3.00	3.75
K	"	1.50	1.73	2.00	2.50

on of **Stresses.** As a general rule, the stresses in a roof-truss can much more readily by the GRAPHIC METHOD than by MATHEMATI-IONS and with as close a degree of accuracy as is necessary. There is of trusses, however, for which the stresses can be more easily COMPUTATION. Such trusses must be symmetrical in shape ads all alike, as is quite frequently the case with simple steel roofs ng-load.

) XIII give constants by which the stresses in Fink and fan readily COMPUTED simply by multiplying the constant by the load. These tables apply, however, only when the rafter is struts into equal spaces, giving equal panel-loads. For any is the stresses should be determined by the GRAPHIC METHOD.

Coefficients for Determining the Stresses in an Eight-Panel Fink Truss

WHEN PANEL-LOADS ARE ALL EQUAL



Eight-panel Fink Truss

stress in any member, multiply its factor by the panel-load, P

Kind of stress	$l/h=3$	$l/h=3.464$ $=30°$	$l/h=4$	$l/h=5$
Compression	6.31	7.00	7.83	9.42
"	5.76	6.50	7.38	9.05
"	5.20	6.00	6.93	8.68
"	4.65	5.50	6.48	8.31
"	0.83	0.87	0.89	0.93
"	1.66	1.73	1.79	1.86
"	0.83	0.87	0.89	0.93
ension	0.75	0.87	1.00	1.25
"	0.75	0.87	1.00	1.25
"	1.50	1.73	2.00	2.50
"	2.25	2.60	3.00	3.75
"	5.25	6.06	7.00	8.75
"	4.50	5.19	6.00	7.50
"	3.00	3.46	4.00	5.00

Table XII. Coefficients for Determining the Stresses in Cambered Fink and Fan Trusses

WHEN PANEL-LOADS ARE ALL EQUAL AND THE CAMBER EQUALS ONE-SIXTH THE RISE

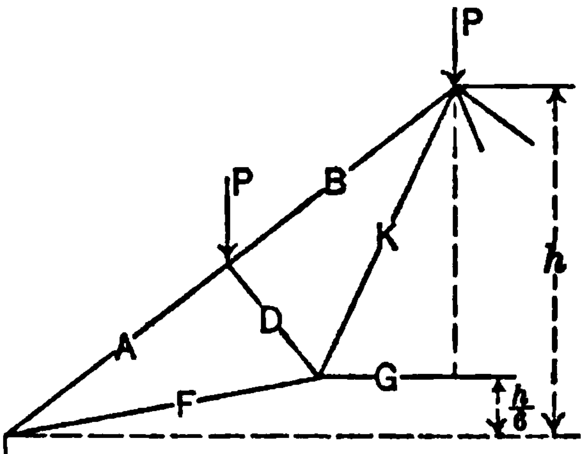


Fig. A

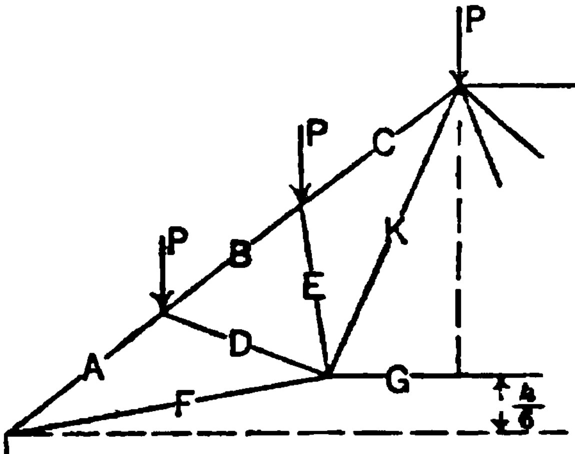


Fig. B

To find the stress in any member, multiply its factor by the panel-load, *P*

TRUSS LIKE FIG. A

Member	Kind of stress	$l/h = 3$	$l/h = 3.464 = 30°$	$l/h = 4$	$l/h = 5$
A...........	Compression	3.64	4.13	4.70	5.78
B...........	"	3.09	3.63	4.25	5.41
D...........	"	0.83	0.87	0.89	0.93
F...........	Tension	3.07	3.62	4.24	5.40
G...........	"	1.80	2.08	2.40	3.00
K...........	"	1.43	1.69	1.98	2.52

TRUSS LIKE FIG. B

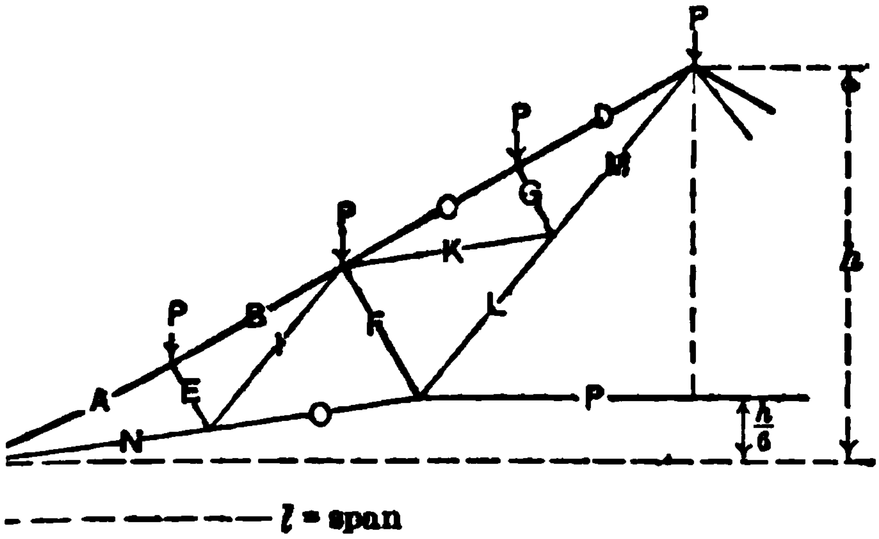
Member	Kind of stress	$l/h = 3$	$l/h = 3.464 = 30°$	$l/h = 4$	$l/h = 5$
A...........	Compression	6.09	6.88	7.83	9.64
B...........	"	4.89	5.63	6.48	8.10
C...........	"	4.96	5.88	6.93	8.89
D...........	"	1.04	1.15	1.26	1.49
E...........	"	1.04	1.15	1.26	1.49
F...........	Tension	5.12	6.03	7.07	9.01
G...........	"	2.70	3.12	3.60	4.50
K...........	"	2.66	3.13	3.67	4.69

Table XIV gives coefficients which are general for any span and depth eight-panel roof-trusses with the Howe and Pratt types of bracing. Tables and XVI give formulas for COMPUTING the stresses in symmetrical Howe Pratt trusses which are symmetrically loaded. The coefficients are given trusses having an odd number of panels. For the Howe truss with an e number of panels the coefficients for the center load on the top chord are divided by two. For the center load on the bottom chord the coefficients also divided by two, except that for the center vertical, which remains un

truss with an even number of panels the coefficients are divided : center loads for all pieces, except that for the center vertical : top chord, the coefficient remains unity. For the young architect se tables will be found useful in furnishing a check upon stresses GRAPHIC METHODS.

Coefficients for Determining the Stresses in an Eight-Panel Cambered Fink Truss

-LOADS ARE ALL EQUAL AND CAMBER EQUALS ONE-SIXTH THE TOTAL RISE



= 3.5 P

: stress in any member, multiply its factor by the panel-load, *P*

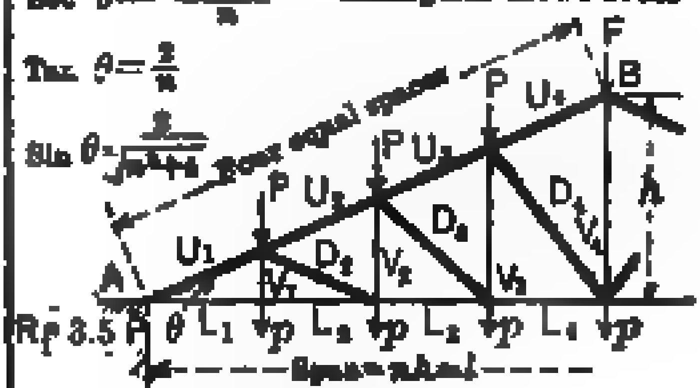
Kind of stress	$l/h=3$	$l/h=3.464$ $=30°$	$l/h=4$	$l/h=5$
Compression	8.49	9.63	10.96	13.49
"	7.94	9.13	10.51	13.11
"	7.39	8.63	10.06	12.74
"	6.83	8.13	9.61	12.37
"	0.83	0.87	0.89	0.93
"	1.66	1.73	1.79	1.86
"	0.83	0.87	0.89	0.93
Tension	1.02	1.21	1.41	1.80
"	1.02	1.21	1.41	1.80
"	2.87	3.37	3.96	5.04
"	3.89	4.58	5.37	6.85
"	7.17	8.44	9.90	12.61
"	6.15	7.23	8.48	10.81
"	3.60	4.16	4.80	6.00

Table XIV. Coefficients for Eight-Panel Roof-Trusses

Sec $\theta = \frac{\sqrt{n^2+4}}{n}$ **Triangular Howe Truss** **Triangular Pratt Truss**

Tan $\theta = \frac{2}{n}$

Sin $\theta = \frac{2}{\sqrt{n^2+4}}$

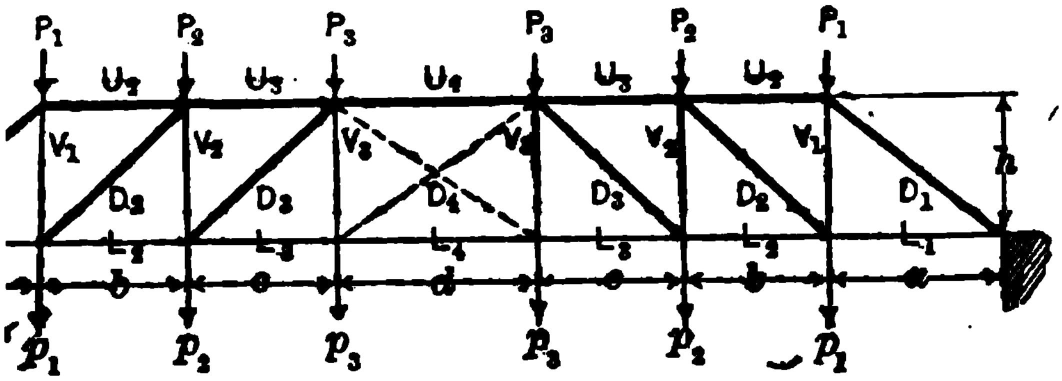


Member	Roof-loads, P	Ceiling-loads, p	Length of member	Roof-loads, P	Ceiling-loads, p	Length of member
U_1	$1.75\sqrt{n^2+4}$	$1.75\sqrt{n^2+4}$	$0.125h\sqrt{n^2+4}$	$1.75\sqrt{n^2+4}$	$1.75\sqrt{n^2+4}$	$0.125h\sqrt{n^2+}$
U_2	$1.50\sqrt{n^2+4}$	$1.50\sqrt{n^2+4}$	"	$1.75\sqrt{n^2+4}$	$1.75\sqrt{n^2+4}$	"
U_3	$1.25\sqrt{n^2+4}$	$1.25\sqrt{n^2+4}$	"	$1.50\sqrt{n^2+4}$	$1.50\sqrt{n^2+4}$	"
U_4	$1.00\sqrt{n^2+4}$	$1.00\sqrt{n^2+4}$	"	$1.25\sqrt{n^2+4}$	$1.25\sqrt{n^2+4}$	"
L_1	$1.75\ n$	$1.75\ n$	$0.125\ nh$	$1.75\ n$	$1.75\ n$	$0.125\ nh$
L_2	$1.75\ n$	$1.75\ n$	"	$1.50\ n$	$1.50\ n$	"
L_3	$1.50\ n$	$1.50\ n$	"	$1.25\ n$	$1.25\ n$	"
L_4	$1.25\ n$	$1.25\ n$	"	$1.00\ n$	$1.00\ n$	"
V_1	0	1.0	$0.25\ h$	1.0	0	$0.25\ h$
V_2	0.5	1.5	$0.50\ h$	1.5	0.5	$0.50\ h$
V_3	1.0	2.0	$0.75\ h$	2.0	1.0	$0.75\ h$
V_4	3.0	4.0	$1.00\ h$	0	1.0	$1.00\ h$
D_2	$0.25\sqrt{n^2+4}$	$0.25\sqrt{n^2+4}$	$0.125h\sqrt{n^2+4}$	$0.25\sqrt{n^2+16}$	$0.25\sqrt{n^2+16}$	$0.125h\sqrt{n^2+}$
D_3	$0.25\sqrt{n^2+16}$	$0.25\sqrt{n^2+16}$	$0.125h\sqrt{n^2+16}$	$0.25\sqrt{n^2+36}$	$0.25\sqrt{n^2+36}$	$0.125h\sqrt{n^2+}$
D_4	$0.25\sqrt{n^2+36}$	$0.25\sqrt{n^2+36}$	$0.125h\sqrt{n^2+36}$	$0.25\sqrt{n^2+64}$	$0.25\sqrt{n^2+64}$	$0.125h\sqrt{n^2+}$

Stress = coefficient × P or p.

For a half-truss supported at A and B, reduce all top-chord coefficients by $\sqrt{n^2+}$ and all bottom-chord coefficients by n. The coefficients for the web-members m[illegible] remain unchanged.

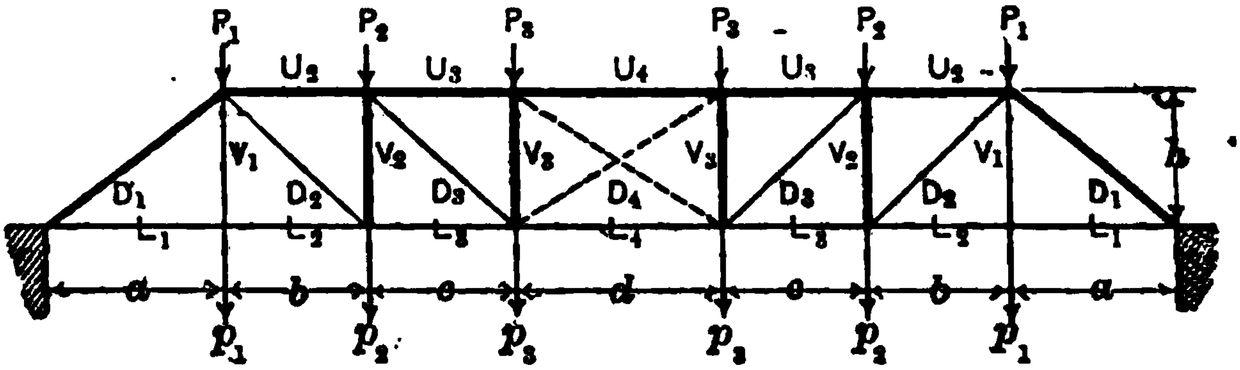
Coefficients for Howe Trusses which are Symmetrical About the Center of the Span and Symmetrically Loaded



7 panels			5 panels		3 panels
P_1	P_2	P_3	P_1	P_2	P_1
$\cdot h$	$a+h$	$a\div h$	$a+h$	$a+h$	$a+h$
$\cdot h$	$(a+b)+h$	$(a+b)+h$	$a+h$	$(a+b)+h$	....
h	$(a+b)\div h$	$(a+b+c)+h$	....	....	....
h	$(a+b)\div h$	$(a+b+c)+h$	....	....	....
$^2+h^2\div h$	$\sqrt{a^2+h^2}+h$	$\sqrt{a^2+h^2}+h$	$\sqrt{a^2+h^2}+h$	$\sqrt{a^2+h^2}\div h$	$\sqrt{a^2+h^2}\div h$
0	$\sqrt{b^2+h^2}+h$	$\sqrt{b^2+h^2}+h$	0	$\sqrt{b^2+h^2}\div h$	....
0	0	$\sqrt{c^2+h^2}+h$	0	0	....
0	0	0	....	....	....
0	1.0	1.0	0	1.0	0
0	0	1.0	0	0	....
0	0	0	....	....	....
p_1	p_2	p_3	p_1	p_2	p_1
.0	1.0	1.0	1.0	1.0	1.0
0	1.0	1.0	0	1.0	....
0	0	1.0	....	....	....

, etc., the coefficients for the chords and diagonals are the same as ls P_1, P_2, etc. The coefficients for the verticals for loads p_1, p_2, etc., upplementary table below the general table. Tension is indicated am by light lines.

Table XVI. Coefficients for Pratt Trusses which are Symmetrical About the Center of the Span and Symmetrically Loaded



Member	7 panels			5 panels		3 panels
	P_1	P_2	P_3	P_1	P_2	P_1
L_1 and L_2	$a \div h$	$a \div h$	$a \div h$	$a \div h$	$a \div h$	$a \div h$
L_3 and U_2	$a \div h$	$(a+b) \div h$	$(a+b) \div h$	$a \div h$	$(a+b) \div h$	....
L_4 and U_3	$a \div h$	$(a+b) \div h$	$(a+b+c) \div h$	....	....	....
$U_4 = L_4$...	....	....	....	....	....	....
D_1.......	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div h$	$\sqrt{a^2+h^2} \div$
D_2.......	0	$\sqrt{b^2+h^2} \div h$	$\sqrt{b^2+h^2} \div h$	0	$\sqrt{b^2+h^2} \div h$	....
D_3.......	0	0	$\sqrt{c^2+h^2} \div h$	0	0	....
D_4.......	0	0	0	....	....	....
V_1.......	0	0	0	0	0	0
V_2.......	0	1.0	1.0	0	1.0	....
V_3.......	0	0	1.0	....	....	....
	p_1	p_2	p_3	p_1	p_2	p_1
V_1.......	1.0	0	0	1.0	0	1.0
V_2.......	0	0	1.0	0	0	....
V_3.......	0	0	0	....	....	....

For loads p_1, p_2, etc., the coefficients for the chords and diagonals are the same given for the loads P_1, P_2, etc. The coefficients for the verticals for loads p_1, p_2, etc. are given in the supplementary table below the general table. Tension is indicat in the truss-diagram by light lines.

ımples Showing Use of Tables in Stress-Computations

an Truss. Example 1. In this example a simple fan truss of ; considered. The distance on centers of trusses is 12 ft. The ss is 9 ft, or $l/h = 4$. The total load per square foot of roof is 40 lb. of rafter is 20 ft, nearly. The panel-load, $P = {}^{20}/_{3} \times 12 \times 40 =$ ıen from Table X,

›wer end of rafter $A = 3\ 200 \times 5.59 = 17\ 888$ lb
ıds of main tie $F = 3\ 200 \times 5.00 = 16\ 000$ lb
:nter of main tie $G = 3\ 200 \times 3.00 = 9\ 600$ lb
races D and $E = 3\ 200 \times 1.08 = 3\ 456$ lb
e $K = 3\ 200 \times 2 = 6\ 400$ lb

ıl Howe Truss. Example 2. (Table XV.) A five-panel Howe lered, for which $h = 6$ ft, $a = 9$ ft, $b = 10$ ft and $c = 12$ ft. Let the ced 10 ft on centers, the roof-load be 40 lb per sq ft and the ceiling- : sq ft. The panel-loads become:

$$\left.\begin{array}{l} 9+10)\ (10 \times 40) = 3\ 800 \text{ lb} \\ 9+10)\ (10 \times 15) = 1\ 400 \text{ lb} \end{array}\right\} = 5\ 200 \text{ lb}$$

$$\left.\begin{array}{l} 0+12)\ (10 \times 40) = 4\ 400 \text{ lb} \\ 0+12)\ (10 \times 15) = 1\ 700 \text{ lb} \end{array}\right\} = 6\ 100 \text{ lb}$$

ı $= {}^{9}/_{6} \times 5\ 200 + {}^{9}/_{6} \times 6\ 100 = 17\ 000$ lb
ı $= {}^{9}/_{6} \times 5\ 200 + {}^{19}/_{6} \times 6\ 100 = 27\ 100$ lb
:/6 $(5\ 200 + 6\ 100) = 20\ 400$ lb
;/6 $\times 6\ 100 = 11\ 900$ lb
o $+ 1\ 400 + 1\ 700 = 7\ 500$ lb
ı lb

esults all values between 50 and 100 have been considered 100.

nation of Stresses in Roof-Trusses by Graphic Methods

.c **Method** is the simplest and in most cases the quickest method · the stresses in a roof-truss; and it has, besides, the additional being applicable to any true truss-form or any arrangement of is also less chance of making a mistake in the GRAPHIC METHOD :thod of NUMERICAL COMPUTATION, as an error in the graphical t always becomes manifest. When the principles are under- ›IAGRAMS can be very quickly drawn, without the aid of books the forms of trusses in common use, the method of drawing the is quite simple; and a careful study of the following examples, ›y a little practice in drawing the diagrams, should enable any ;htsman, or builder to understand the principles involved in the .LYSIS OF ROOF-TRUSSES.

Ipon Which the Graphic Method is Based. To thoroughly ; method, a knowledge of the COMPOSITION AND RESOLUTION OF ›lained in Chapter VI, is essential; and before studying this dent should read carefully pages 288 and 289. The theorems ›lained on these pages form the basis of GRAPHIC STATICS. IC METHOD all forces, including the loads, are represented :s, and the directions of the forces must be constantly kept in is of assistance to indicate the direction of a force by an arrow- ıed on page 289. The direction in which a force acts with refer- indicates, also, whether it is a PUSHING or a PULLING force, or nber on which the force or in which the stress acts is in COMPRES- r. This is more fully explained in the following pages, and also ith several of the stress-diagrams.

Forces and Stresses which Act On and In a Truss. Every stress-di gram represents three sets of forces, viz., the external LOADS, the supporti forces or REACTIONS, and the STRESSES in the truss-members.

Supporting Forces or Reactions. For a truss to remain in place, two the conditions for equilibrium are that the algebraic sums of the vertical a horizontal components of all the forces acting upon the truss must respective equal zero. Then the horizontal and vertical components of the supporti forces or reactions, taken together, must respectively equal the horizontal a vertical components of the loads. The LOADS and REACTIONS are consider as the EXTERNAL FORCES acting on the truss and form part of the STRESS-DIAGRA

Symmetrical Loads. When the loads or vertical forces are symmetrical each side of the middle of the span, the supporting forces are equal, and each equal to one-half the total load on the truss.

Unsymmetrical Loads. When the loads are not symmetrical about t middle, either in regard to point of application or to magnitude, the supporti forces are unequal and in most cases must be determined before the stress-di gram can be drawn. The supporting forces for unsymmetrically loaded trus may be computed by the method of the MOMENTS OF FORCES, explained pages 322 to 324.

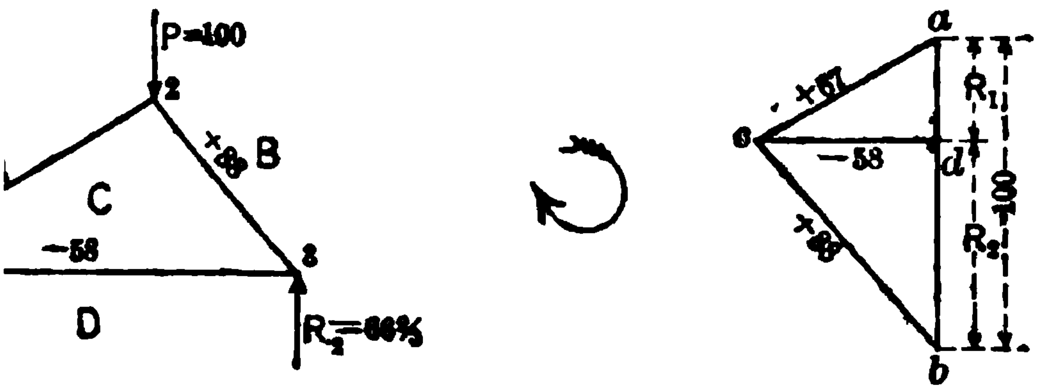
Stress-Diagrams for Vertical Loads. Before the stress-diagram for a tr can be drawn, it is necessary to make a skeleton drawing of the truss, representi the central or median lines of the members as explained on page 1058. T diagram, called the TRUSS-DIAGRAM, should be drawn on the same sheet paper as the STRESS-DIAGRAM, for convenience in drawing the latter. T truss-diagram should also have all of the loads which come on the truss indicat by arrows and figures, as in the following examples.

Supporting Forces. The SUPPORTING FORCES, also, should be indicated the TRUSS-DIAGRAM as in Fig. 10. These forces are determined as explain on pages 322 to 324.

Lettering the Truss-Diagram. After the truss-diagram is drawn, it is co venient to letter it according to the method known as Bow's NOTATION, whi allows a ready comparison of the TRUSS-DIAGRAM and the STRESS-DIAGRA and also enables the student to readily draw the stress-diagram and to imme ately determine the CHARACTER as well as the MAGNITUDE of the stresses. T essential principle of this method is the LETTERING of each space on each s of every external force and of every member of the truss, so that on the tru diagram a truss-member or external force is denoted by the letters on ea side of it. When the stress-diagram is drawn, it will be found that the sa letters come at the ends of the lines representing the external forces and t stresses in the truss-members.

The Simple Triangular Frame is much used in building construction, a most forms of roof-trusses are combinations of such triangles. It is, therefo worth while to show how easily the above principles may be used to determi the stresses in such a frame. Diagram 1, Fig. 6, represents the TRUSS-DIAGR of a triangular frame properly lettered. A load of 100 lb is applied at the ap The weight of the frame is disregarded. In diagram 2, a vertical line *ab* is draw 1 in long (say to a scale of 100 lb to the inch), representing the force A From *b*, *bd* is drawn equal to R_2 and from *d*, *da* equal to R_1. These three li represent the external forces acting on the truss, and the polygon *abda*, cal the FORCE-POLYGON, is always a CLOSED FIGURE if the forces are in EQUILIBRIU Since the force *AB* is vertical and R_1 and R_2 are parallel to *AB*, the figure be a straight line, *bd* and *da* coinciding with *ab*. If the external forces form a CLOS

:n laid off to scale, usually in order, the frame or truss upon which
not be moved either vertically or horizontally by the forces. The
ON should always be drawn and closed before any attempt is made
the stresses in the members of the truss. The stresses in the
he truss will now be found, beginning with those meeting at joint 1.
d *CD* meet at this joint. The stresses in these two pieces and R_1 are
RM and, consequently, if laid off in order will form a CLOSED FIGURE
Chapter VI. In diagram 2, *da* represents R_1 in MAGNITUDE and
From *a* draw a line parallel to *AC* and from *d* a line parallel to *CD*



l. Truss-diagram 2. Stress-diagram

Fig. 6. Triangular Frame

iem until they intersect at *c*. *ac* is the stress in *AC*, and *cd* that
and *da*, or R_1, are in EQUILIBRIUM since they form a CLOSED FIG-
the forces in order, *da*, or R_1, is known to act towards the joint.
ac is also towards the joint and hence the stress is of the same
ie force R_1 and the piece *AC* is in COMPRESSION. *AC* pushes
it as R_1 does. Continuing around the stress-polygon *dac*, in the
cd acts away from the joint and the stress in *CD* is opposite in
ie force R_1, or *CD* is in TENSION. *CD* pulls away from joint 1.
stresses in the pieces *BC* and *CA* and the force *AB* are in EQUI-
sides of the STRESS-POLYGON are *ab*, *bc* and *ca* (diagram 2). The
represents the load of 100 lb acts down and towards the joint,
ict towards this joint, showing that the stresses in *BC* and *CA*
character as the force *AB*, or that the pieces push against the
each is in COMPRESSION. At joint 3, the two pieces meeting are
The STRESS-POLYGON is *bdc*. Here *bd* acts towards the joint, *dc*
joint, and *cb* towards the joint. As found before, the stress in
and that in *CB*, COMPRESSION. Diagram 2 is made up of three
rs, one for each of the joints shown in diagram 1. Each of these
sidered independently when determining the MAGNITUDE and
he stresses or forces. This is important to remember when the
s are combined as in diagram 2. In determining the CHARACTER
4*C*, for example, from the STRESS-POLYGON *dac* for joint 1, the
vards joint 1, while from the STRESS-POLYGON *abc* for joint 2, *ca*
at 2. In both cases the piece *AC* is pushing against the joints
s in COMPRESSION. If arrow-heads are used in indicating the di-
orces in the STRESS-POLYGONS, they should be erased as soon as
f the stresses for the joint being considered have been found;
polygons are combined as in diagram 2, each line will have two
iting in opposite directions, leading to confusion. Arrow-heads
on the TRUSS-DIAGRAM. Each piece will have two arrow-heads,
referring to the joint at the end. When the arrow-heads point

away from each other the piece is in COMPRESSION, and when they po towards each other the piece is in tension.

It is important to keep in mind the direction in which the forces and stres are considered in order, in going around the truss or around a joint. In Fi 6 and 8 the curved arrows show that a clockwise direction has been chos This makes the stress-lines of the stress-diagram come on the left of the lo line. This direction has been taken for all the trusses in this chapter, exc for a few diagrams for wind-loads. The stres could have been determined just as well taking a contra-clockwise direction.

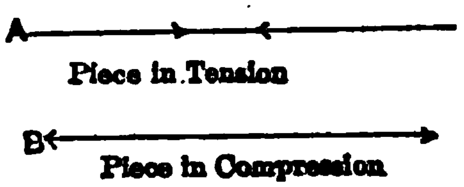
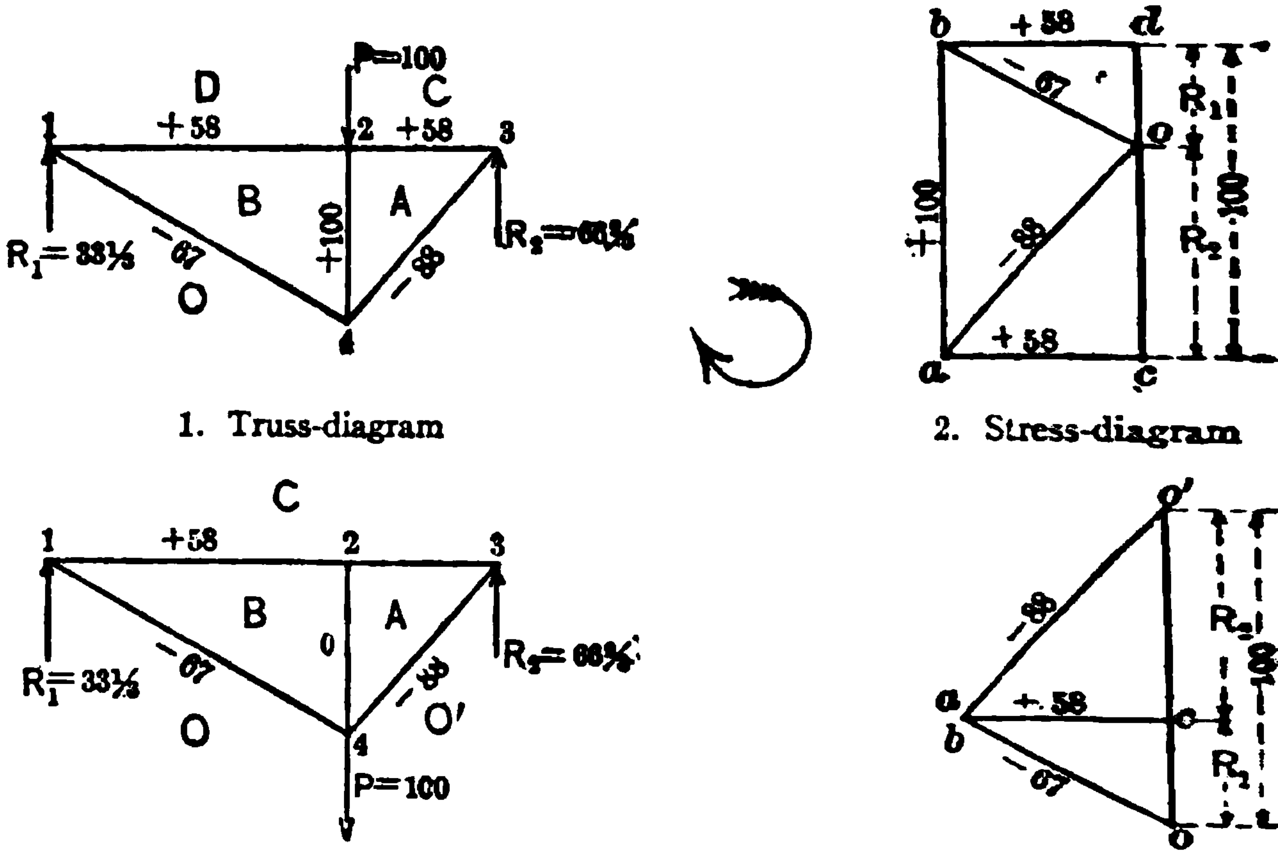


Fig. 7. Indication of Character of Stress

If two men pull on the two ends of a ro exerting PULLING FORCES of equal intensity, TENSIONAL STRESS in every cross-section of rope is equal to the FORCE with which one m pulls; and each end of the rope pulls away fr the man holding it, with a FORCE equal in magnitude to that which he exe Thus if each man exerts a FORCE of 100 lb the STRESS in the rope is 100 lb each end of the rope pulls away with a FORCE of 100 lb. If the men push agai the two ends of a piece of timber with a FORCE of 100 lb, the timber pus against each man with a FORCE of 100 lb, although the entire COMPRESS STRESS in every cross-section of the timber is but 100 lb. Consequen STRESS-LINES are sometimes drawn with arrow-heads pointing towards e other, as at A, Fig. 7, denoting TENSION; or with arrow-heads point



1. Truss-diagram 2. Stress-diagram

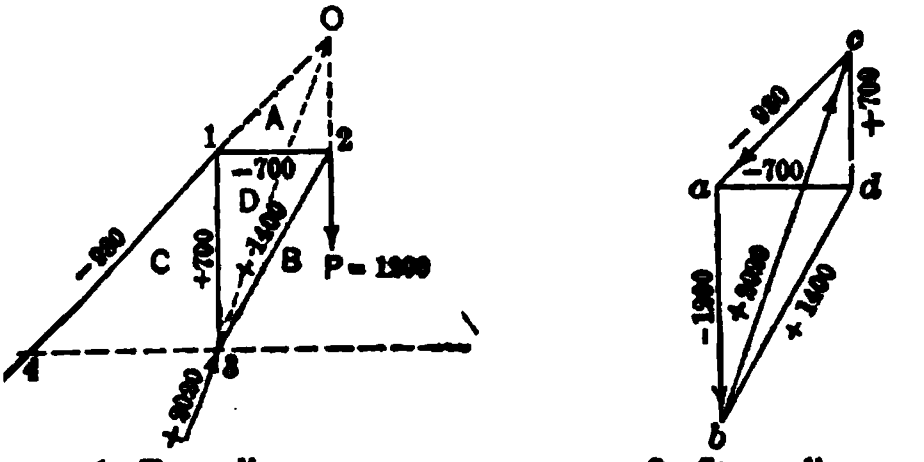
3. Truss-diagram 4. Stress-diagram

Fig. 8. Trussed Beam

in opposite directions, as at B, denoting COMPRESSION. It is better, howe to omit arrow-heads on STRESS-LINES, putting them on lines represen EXTERNAL FORCES only. The STRESS in any member of a truss act opposite directions at the two ends of the piece. This is an important t to remember in drawing STRESS-DIAGRAMS.

The Trussed Beam. Fig. 8 shows a load supported by a beam, post or s and two ties instead of by two struts and a tie. The effect on the rod form

s the same whether the load is applied as shown in diagram 1, or iagram 3. Considering the case shown in diagram 1: The FORCE-*od* (diagram 2); the sides of the STRESS-POLYGON for joint 1 are *od*, e stress in *DB* being compression, and that in *BO*, tension. For les of the stress-polygon are *dc*, *ca*, *ab* and *bd*, the stress in *CA* be-on; that in *AB*, compression; and that in *BD*, compression. For es of the stress-polygon are *ac*, *co* and *oa*. The stress in *AC* is com- that in *OA*, tension. The condition shown in diagram 3, where



1. Truss-diagram 2. Stress-diagram

Fig. 9. Crane Truss

ended from joint 4, leads to a different form of STRESS-DIAGRAM, d of construction remains the same. The stresses in the pieces ith the exception that the stress in the piece *AB* is zero for the case am 3.

Truss. Fig. 9, diagram 1, shows the TRUSS-DIAGRAM of a CRANE ical load at joint 2. The EXTERNAL FORCES acting on the frame t joint 2, the supporting force at joint 3, and the stress in the the frame is in equilibrium under the action of these three forces, a point.
a ready
ermining
d magni-
ipporting
Prolong
d draw a
through
ntersects
onged, at
the sup-
ust pass
, 3*O* is
of this
E-POLY-

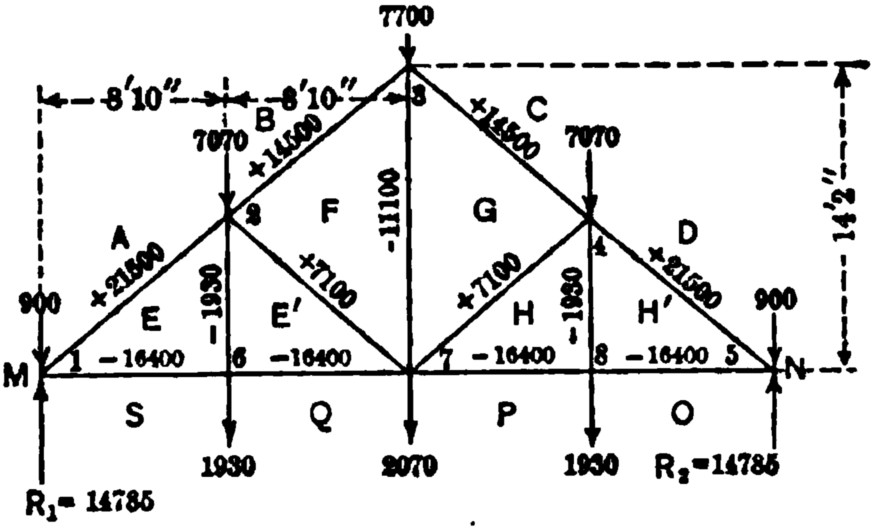


Fig. 10. King-rod Truss. Truss-diagram

n. The sides of this polygon are *bc*, *ca* and *ab*. *ca* is the stress hich is in tension. The stress-polygons for each joint can now n and the stresses in the members of the frame determined

EXAMPLES, worked out in detail and with considerable repeti- the student to grasp the principles of the GRAPHIC METHOD TRESSES IN FRAMED STRUCTURES.

uss. Example 1. Fig. 10 shows the TRUSS-DIAGRAM of the in Fig. 1, properly drawn, lettered and figured, ready for

drawing the STRESS-DIAGRAM. The supporting force at the left is SM, the lo at joint 1 is MA, the bottom of the rafter is AE, the left portion of the tie-bea or bottom chord is ES, etc. The loads acting at joints 1, 2, 3, 4 and 5 a designated as MA, AB, BC, CD and DN respectively, and those at joints 8, and 6 as OP, PQ and QS respectively. It makes no difference what lette are used, except that it is better to first letter the outside spaces consecutive and then the inside spaces.

Force-Polygon. The FORCE-POLYGON is now constructed by laying off scale (Fig. 10A) the external forces in order, beginning with the force MA, a following with AB, BC, CD, DN laid downward, NO laid off upward, OP, P QS laid off downward, and SM laid upward. If the work is correct, the forces form a CLOSED FIGURE.

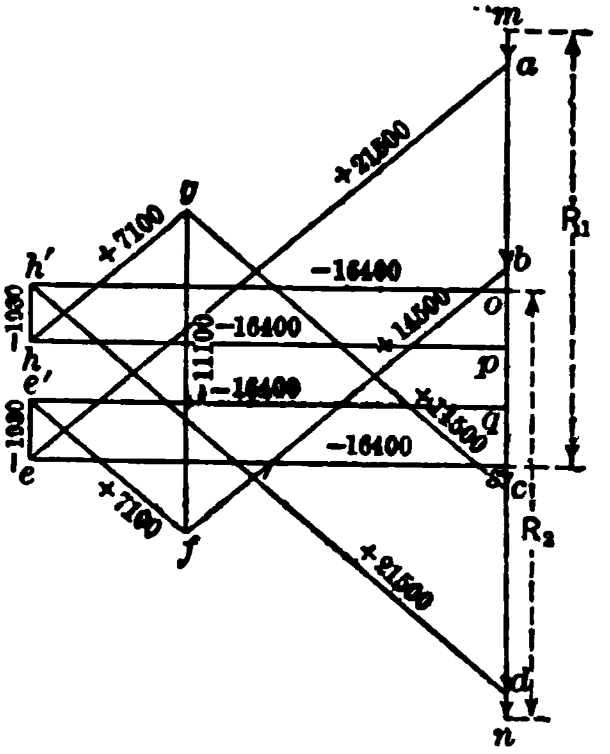


Fig. 10A. King-rod Truss. Stress-diagram

Stress-Diagrams. The STRESS-DI GRAM is drawn by taking the forces acti on the joints in consecutive order, co mencing at one of the supports. It convenient to start with the support the left, or at joint 1. In actual co putations it is not necessary to numb the joints, but in order to refer to the in the description it is necessary to nu ber them in the illustrations. Commen ing at joint 1, the first step in drawi the STRESS-DIAGRAM is to draw a vertic line to a scale of POUNDS-TO-THE-INCH represent the supporting force SM. Th line is the line sm already drawn in co structing the FORCE-POLYGON (Fig. 10 which might be drawn to the scale of 16 000 lb to the inch. It is best use a scale as large as convenient in order to have a relatively small STRES DIAGRAM. An engineer's scale, one divided to 10ths, 20ths, 30ths, etc., of inch, is found most convenient for these drawings. The small letter s is plac at the bottom of the line sm, and the letter m at the top. From m is laid off representing the load MA. A line is then drawn from a, parallel to the raft AE, and a line from s parallel to the tie-beam ES. The two lines meet at and ae represents the stress in AE and es the stress in ES. The supporting for represented by sm, acts upward, and the others follow in rotation, showing th ae acts towards joint 1 and that the member AE is in COMPRESSION, and sho ing that es acts from joint 1 and that the member ES is in TENSION. Consid next the stresses at joint 6. Commencing at the bottom of the joint and goi around to the left, the first force that is known is the load QS, or 1 930 lb, whi is measured to the scale used from q to s, downward, as the loads act downwa The point s was located in drawing the STRESS-POLYGON for joint 1, and q an in constructing the FORCE-POLYGON for the external forces. The next st that is known is the stress se which has just been determined. As this st acts to the right from joint 1, it will act to the left from joint 6, as the stre in the two ends of a strut or a tie act in opposite directions, as explained on p 1068. The stresses in EE' and $E'Q$ are not known, so from e a line is dra parallel to EE' and extended so that a line drawn from its extremity e' parallel to $E'Q$ closes on q. The lines ee' and $e'q$ are thus found, which rep sent the stresses in EE' and $E'Q$ respectively. Starting with se, the stres

be TENSION and acting from joint 6, and going around the diagram E' and E'Q are found to be in TENSION. At joint 2 the stresses A and the force or load AB are known, leaving the stresses in BF e determined. From a lay off downward ab equal to the force or rom b draw a line parallel to BF, and from e' a line parallel to g these lines until they intersect at f; then bf is the stress in BF n FE'. Both members are in COMPRESSION. At joint 3, the un- or stresses are the stresses in CG and GF. From c draw a line , and from f a line parallel to GF. The two lines intersect at g, stress in at in GF. MPRESSION TENSION. uss is sym- symmetri- the stresses rs on the middle are n those on stresses in m the left have been that it is

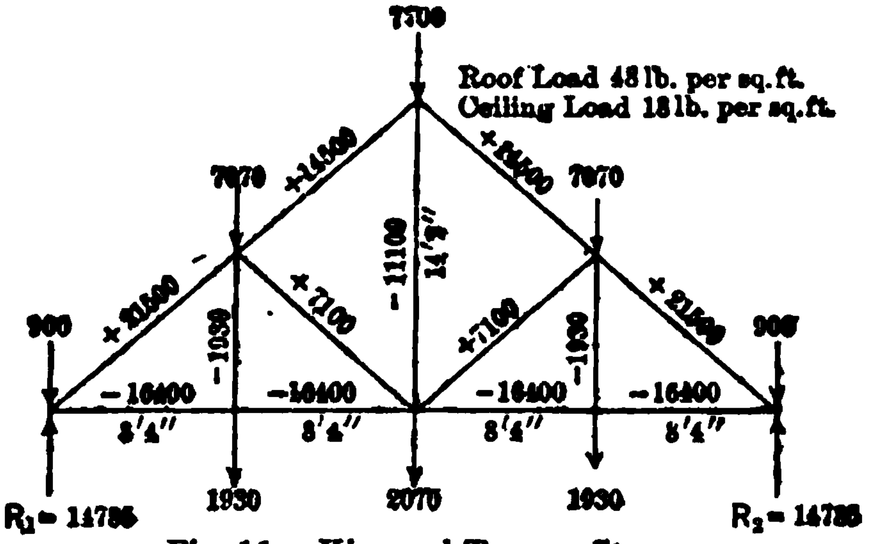
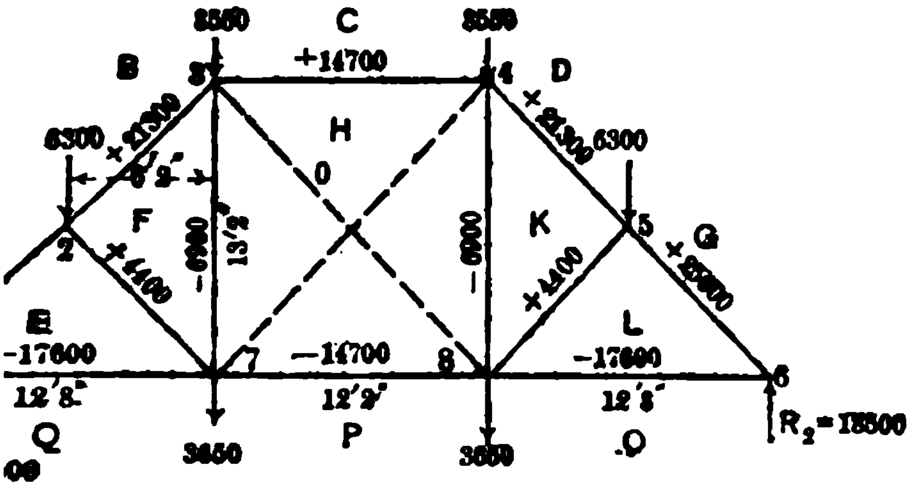


Fig. 11. King-rod Truss. Stresses

to draw the STRESS-POLYGONS for joints 4, 5, 8 and 7. It e to complete the STRESS-DIAGRAM including the stresses for the truss. A closed symmetrical figure will result, unless some n the construction, thus checking the work. The scale is now different lines of the STRESS-DIAGRAM and the MAGNITUDES OF btained as indicated on the corresponding lines of the TRUSS-DIA-

In practice the diagrams of Figs. 10 and 11 are combined in



russ. Truss-diagram. (See, also, Figs. 3, 53 and 54 and Chapter XXVIII, Fig. 1)

hey are shown separately here merely to indicate the succes- lrawing of the diagrams and in the determination of the stresses.

Truss. Example 2. The diagram in Fig. 12 represents the he members of the QUEEN TRUSS shown in Fig. 3; and the re those found in example 2, page 1055. The middle braces in are indicated by dotted lines in the truss-diagram, because ical load there are no stresses in these members, and they are resented by lines in the stress-diagram. As the truss is syr

metrically loaded, each supporting force or reaction is equal to one-half the tot load, or 18 500 lb. There are no purlins at joints 1 and 6 to carry rafters an ceiling-joists, which are supported by the walls of the building, so there are n external loads at these joints as in the previous example. The very small de load due to the truss itself is neglected. To draw the force-polygon, first dra the vertical line *qa* (Fig. 12A) equal in length, to some scale, to the magnitu of the left supporting force; then in rotation and at the same scale lay off tl distances *ab*, *bc*, *cd* and *dg*, downward; *go*, equal to the right supporting forc upward; and *op* and *pq* downward, closing the figure at *q*. To construct tl combined stress-diagram using the force-polygon just drawn, as a foundatio first consider joint 1. From *a* draw a line parallel to *AE* and from *q* a li parallel to *EQ*. The triangle *qae* represents tl three forces in equilibrium, meeting in a point an acting at joint 1. As the supporting forces a upward, the arrow-head on *qa* points upward. Fo lowing the sides of the stress-polygon *qae* in rotatio *ae* acts towards the joint and *eq* from the join showing that *ae* is in compression and *eq* in tensio Next determine the stresses acting at joint 2. Tl stress in *EA* is now known and represented by tl line *ea*, and as the stress at joint 2 acts in a directi opposite to that at joint 1, it now acts upwa towards joint 2. The next force is the load, 6 3 lb, which acts downward. The point *b* has alread been found by measuring from *a* a distance equal 6 300 lb at the same scale as used in drawing *q* There now remain two stresses to be found for joi 2, those in *BF* and *FE*. Draw *bf* parallel to *BF*, an *fe* parallel to *FE*, the two lines intersecting at .

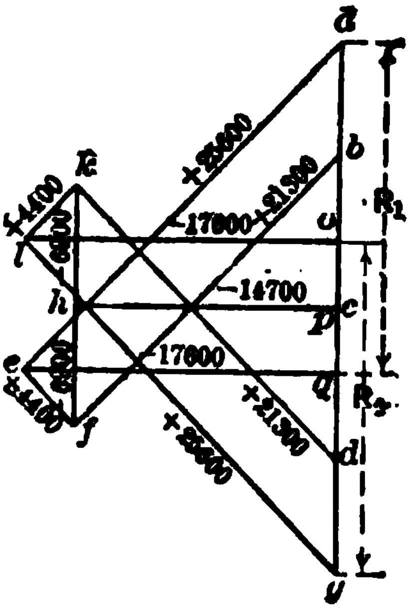
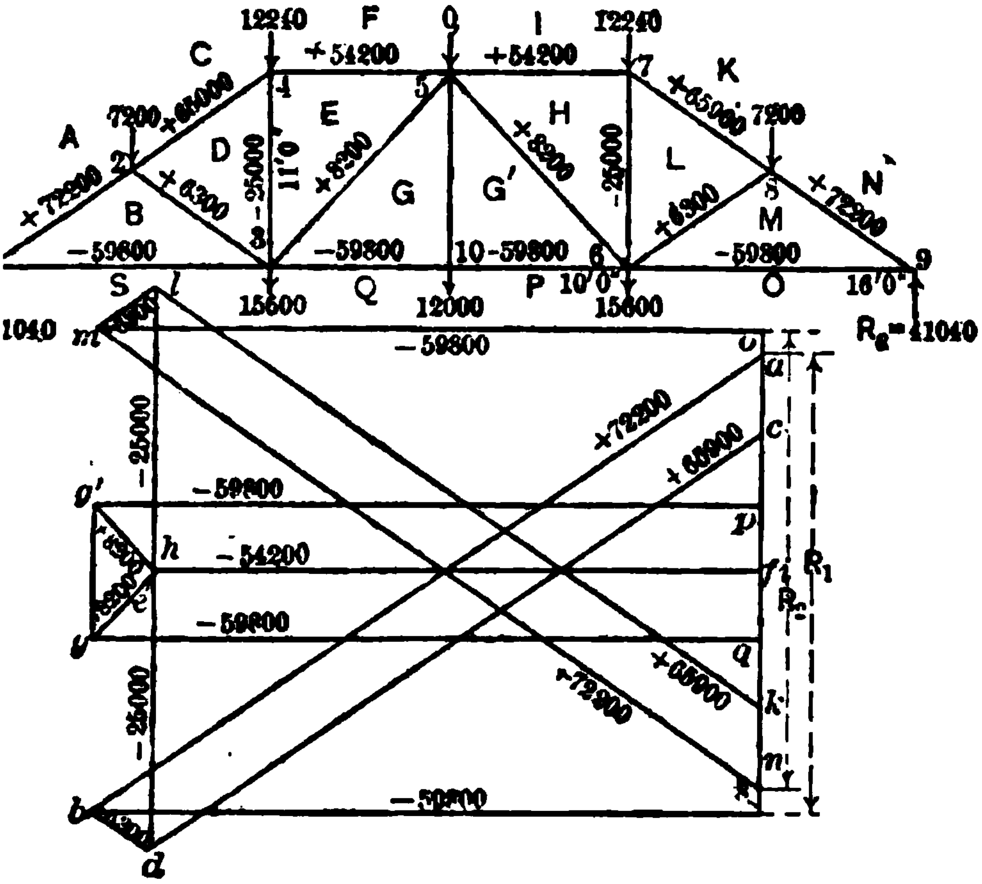


Fig. 12A. Queen Truss. Stress-diagram

Then the sides of the polygon *abfe* represent respectively the magnitud of the four forces acting at joint 2; and the character of the stresses determined by the directions in which the stress-lines are drawn, in orde in going around the joint. In this case they all act toward the join and *EA*, *BF* and *FE* are in compression. The stresses acting at joint 3 or may be determined next, as only two of them are unknown at either join Considering the external force and the three stresses acting at joint 3, the stre in *FB* has been determined and is represented by the line *fb*, which is drawn u ward for this joint. The load or force *BC*, 8 550 lb, is known and is represente by *bc*. *ch* is drawn parallel to *CH*, and *hf* parallel to *HF*, closing the polygo The length of *ch* determines the magnitude of the stress in *CH* and *hf* the stre in *HF*. The stresses in all the truss-members but *HP* are now determine This stress is found by considering the force and stresses acting at joint 7. A this joint the force *PQ*, or 3 650 lb, and the stresses in *QE*, *EF* and *FH*, represente respectively by *pq*, *qe*, *ef* and *fh*, have been determined. The line *hp*, represen ing the stress in *HP*, completes the polygon for joint 7. Hence *hp* determin the stress in *HP*, and as *ho* is drawn from left to right, from the joint, *HP* is i tension. With reference to joint 3, the line *ch* is drawn towards joint 3 an hence *CH* is in compression. Scaling the lines in the stress-diagram (Fig. 12A the figures shown by the side of the lines are obtained. They indicate the ma nitude of each stress in pounds, the + sign denoting compression, and the − sig tension. The two foregoing examples illustrate the method of drawing th stress-diagrams for simple symmetrical trusses, symmetrically loaded. Th truss-diagrams should be drawn in accordance with the measurements give

le of not less than ⅛ in to the foot; and the stress-diagram should ine by line, in accordance with the foregoing directions and the ined and compared with those given in the figures. A variation of lb for small stresses and less than 1% for large stresses may be ex- a greater variation indicates either that sufficient care has not been drawing the stress-lines exactly parallel to the corresponding lines diagram, or that an error has been made in drawing the truss-dia- scaling the lines of the stress-diagram. After these two examples orked, a number of the following examples, also, should be solved, nciples are fully understood.

r **Museum of Fine Arts, St. Louis, Mo. Example 3.** Fig. 13 ne truss-diagram of the truss shown in Fig. 11, Chapter XXVI,



. 13. Truss-diagram. Museum of Fine Arts, St. Louis, Mo.
Fig. 13A. Stress-diagram

ated being approximately those due to the roof and suspended The loads being symmetrically disposed, each supporting force is alf the total load, or 41 040 lb. The counterbraces *CC*, shown VI, are omitted from the truss because they have no stress when niformly loaded. To draw the stress-diagram (Fig. 13A), first he vertical line *sa*, equal to 41 040 lb, equal to R_1; and then *ab* respectively to *AB* and *BS* and representing the stresses acting joint 2, the line *ba* represents the stress in *BA*; *ac*, equal to 7 200 ; *cd*, the stress in *CD*; and *db* the stress in *DB*. The polygon s the forces in equilibrium acting at joint 2. At joint 3 there are forces; and as three unknown forces out of five in one polygon mined, joint 4, where *dc* and the load *CF* are known, is considered ing off the load *cf*, equal to 12 240 lb, the stresses in *FE* and *ED* determined. These are found by drawing *fe* parallel to *FE*, and *D*, the two lines intersecting at *e*. At joint 3, *sb*, *bd*, *de* and the own, and *eg* and *gq* are drawn to close the polygon *sbdegs*. At

joint 10 the force *pq*, equal to 12 000 lb, and *qg* are known; and *gg'* and *g'p* a drawn to close the polygon. At joint 5, *g'g*, *ge* and *ef* are known and *ih* and *h* are drawn to close the polygon Since there is no load at joint 5, *f* and *i* fall the same point in the stress-diagram. The stresses in pounds, in the vario members of the truss, are given in numbers on the corresponding lines in th stress-diagram (Fig. 13A).

Triangular Howe Truss. Example 4. Consider the skeleton triangul Howe truss represented in Fig. 14 loaded as shown by the weight of the ro

F

above and a ceiling below. To draw the stress-diagram, first draw to sc the supporting force *qj*, equal to 46 620 lb. Then lay off *jk* equal to 10 320 *kl* equal to 10 320 lb, etc. Then draw the lines *ja* and *aq*, and the three for at the left support are known. At joint 0, *pq* and *qa* are known and *ab* and are drawn to close the polygon. At joint 1, *ba*, *aj* and *jk* are known and and *cb* are drawn to close the polygon. At joint 2, *op*, *pb* and *bc* are kno and *cd* and *do* are drawn to close the polygon. At joint 3, *dc*, *ck* and *kl* are kno and *le* and *ed* are drawn. At joint 4, *no*, *od* and *de* are known and *ef* and *fn*

se the polygon. At joint 5, *fe*, *el* and *lm* are known and *mg* and *gf*
Joint 7 is considered next, for at joint 6 there are three unknown
d by the graphic method three out of five forces, meeting in a point
ibrium, must be known in order to determine the other two. At
nd *mm'* are known and *m'g'* and *g'g* are drawn to close the polygon.
tes the determination of the stresses in all the pieces for one-half
and of course the stresses for each half are the same as the loading
al.

nel Howe Truss. Example 5. For the next example a Howe
isidered, whose center lines give the diagram shown in Fig. 15.

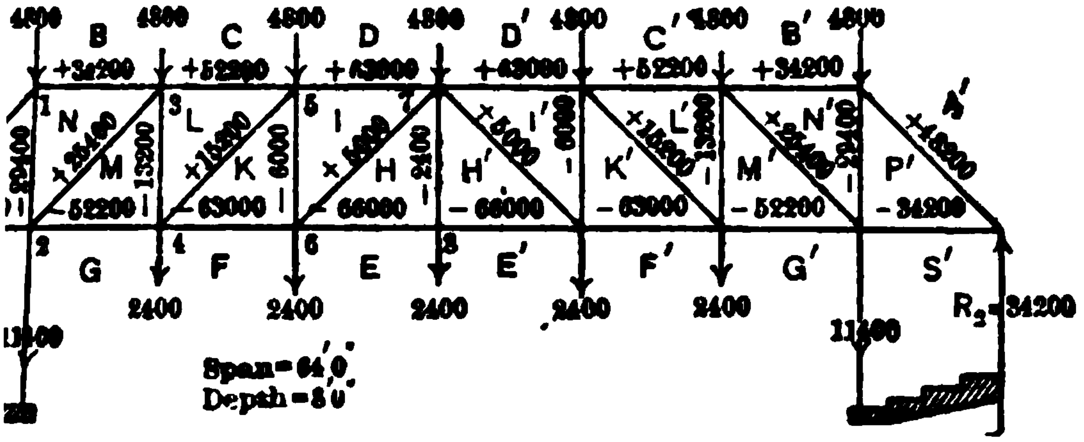


Fig. 15. Howe Truss. Truss-diagram

ised for a span of 64 ft, and it supports, in addition to a flat roof,
ng below the bottom chord and a gallery on each side. The loads
at joints are about as indicated in Fig. 15. To draw the stress-
15A), first construct the force-polygon by laying off to scale in
xternal forces, commencing with the left reaction 34 200 lb. Next,
t joint 0, the supporting force *sa* is known, the stress in the rafter
stress in the tie *ps*, closing the polygon. At joint 1, *pa* and *ab*

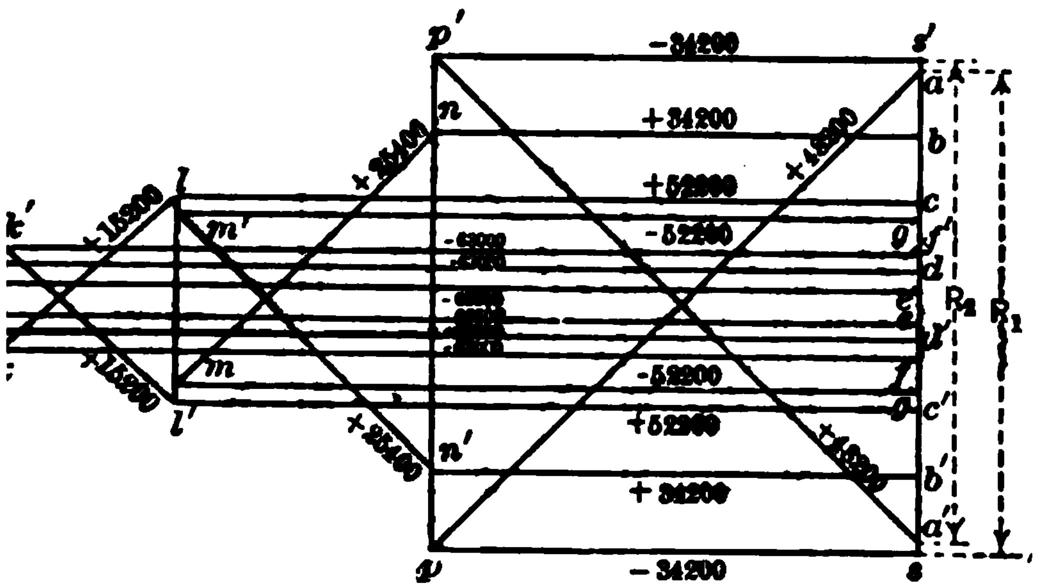


Fig. 15A. Howe Truss. Stress-diagram

bn and *np* are drawn, closing the polygon. At joint 2, *gs*, *sp* and
nd *nm* and *mg* are drawn. At joint 3, *mn*, *nb* and *bc* are known
re drawn. The stresses at the remaining joints are found in the
ose at 3 and 4. The stresses in pounds in the various members
noted in figures in the stress-diagram (Fig. 15A).

Loaded at Alternate Joints. Example 6. (Fig. 16.) This
owe truss is selected to show how to proceed when there is no

load at one or more of the joints. Fig. 16 represents the center lines of a trus 50 ft in span and only 5 ft in height. In order to give the braces an inclinatio approximating 45°, the truss is divided into ten panels; but purlins are place over every other joint, as in Fig. 16, Chapter XXVI. The loads from thes purlins are about 5 000 lb. The stresses at joint 1 are found in the same mann as in the previous example, always starting with the supporting force. A joint 2 the stress-line *da* is already drawn; and as there is no load at this joint

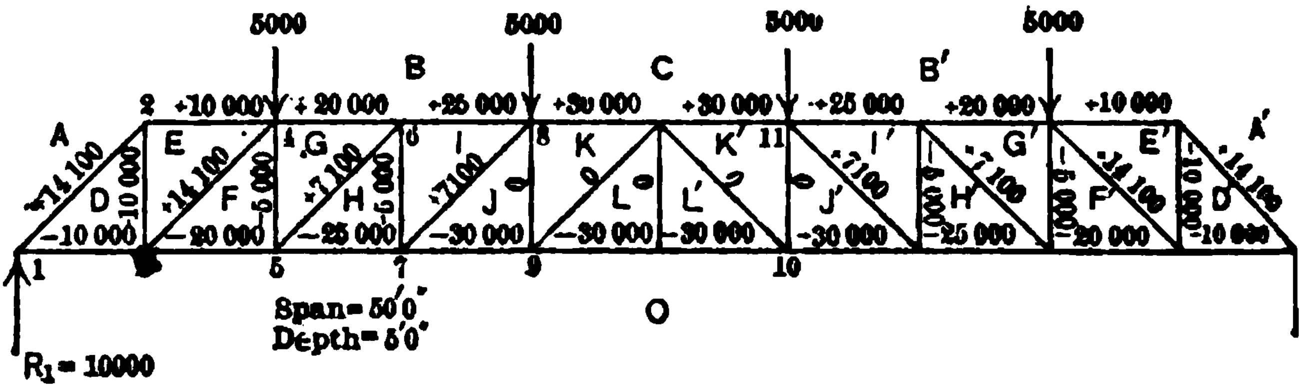


Fig. 16. Howe Truss. Truss-diagram

a line is drawn from *a* parallel to *AE* (*A* covers the entire space from joint to joint 4), and a line from *d* parallel to *ED*, the two lines intersecting at *e*.

The force-lines and stress-lines are as follows:

At joint 3: *od*, *de*, *ef* and *fo*;
At joint 4: *fe*, *ea*, *ab*, *bg* and *gf*;
At joint 5: *of*, *fg*, *gh* and *ho*;
At joint 6: *hg*, *gb*, *bi* and *ih*;
At joint 7: *oh*, *hi*, *ij* and *jo*;
At joint 8: *ji*, *ib*, *bc* and *ck*;

the latter line extending to the point of beginning, *j*, showing that there is n stress in *kj*. At joint 9 the only stresses are *oj* and *lo*, for as there is no stre

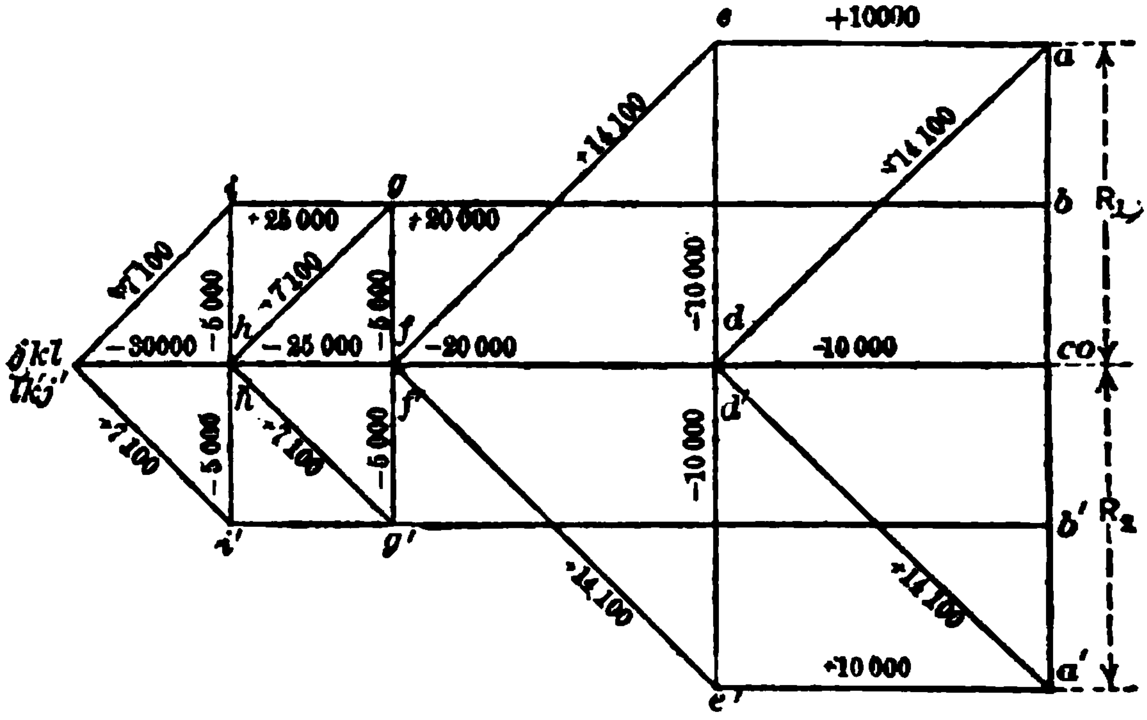
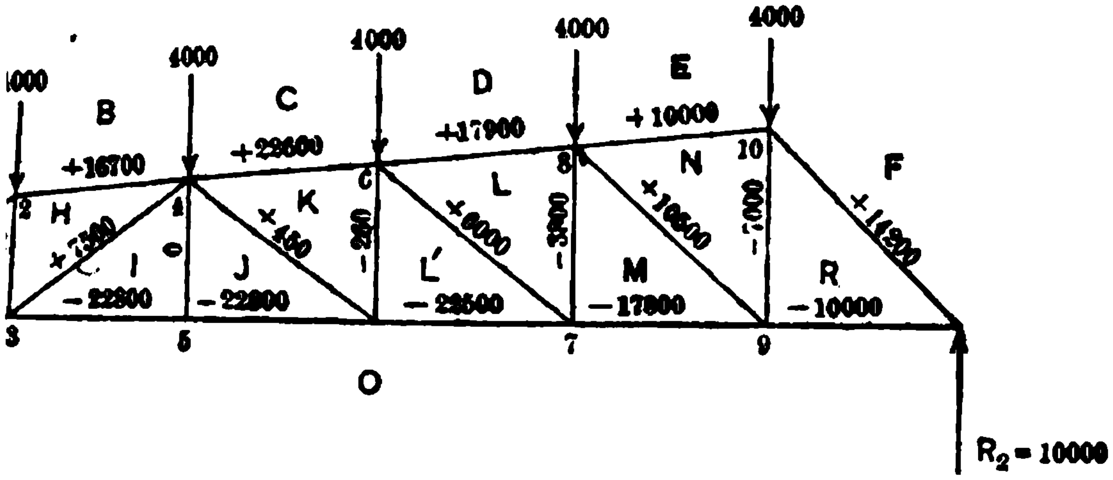


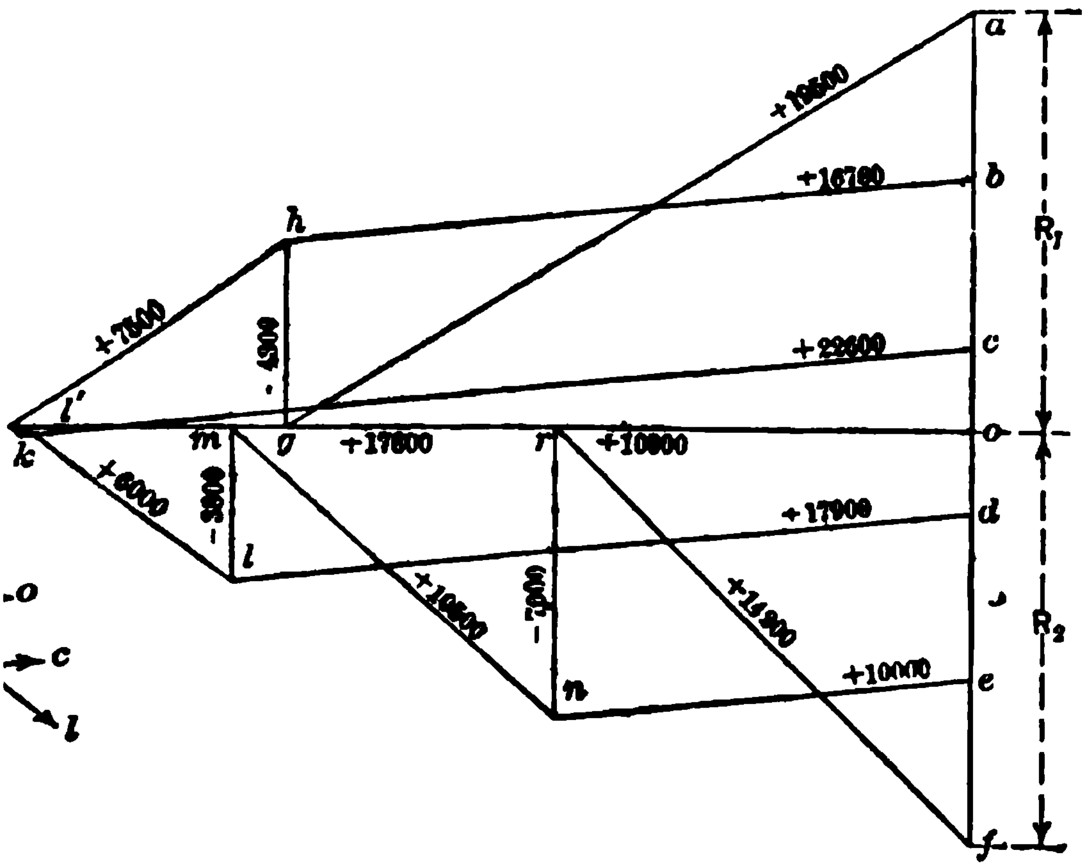
Fig. 16A. Howe Truss. Stress-diagram

in *JK*, for equilibrium there can be none in *KL*. There is, also, no stress in th middle rod. Although these members have no stress, it is advisable to inse them in the truss in order to stiffen the top and bottom chords. They m be made very light, say ½ in in diameter for the rods and 3 by 6 in in cro section for the braces.

russ with Slanting Top Chord. **Example 7.** In order to give a roof it is often desirable to incline the top chord of a HOWE TRUSS ', Chapter XXVI. Fig. 17 shows the truss-diagram for such a truss, A the stress-diagram. The latter is drawn in the same way as the ım in Example 5, but because the top chord is not level, the stress- ıot symmetrical. When the stress-diagram is not symmetrical it is complete the entire diagram, so as to show the stress in every mem- ıss. The stress-lines for joint 9 are *om*, *mn*, *nr* and *ro*. This leaves



. 17. Howe Truss with Slanting Top Chord. Truss-diagram



ı. Howe Truss with Slanting Top Chord. Stress-diagram

:o complete the diagram; and if the diagram has been correctly ning *r* and *f* will be exactly parallel to *RF*. There is no stress

Inclined Ties. **Example 8.** (Fig. 18.) This truss has the as the truss shown in Fig. 14, but the diagonals incline in the n and are in tension, while the verticals, except the middle one, ›ression. This form of truss is sometimes used in wooden con- d the long middle braces shown in Fig. 14. Long ties are, as a

rule, more economical than long struts. The construction of the stress-diagr requires no additional explanation after that given for the stress-diagram Fig. 14A. The student should compare the magnitude of the stresses scal and marked in Fig. 18A with those in Fig. 14A, and note the effect of the chan in the direction of the braces. The truss represented by Fig. 14 requires a ve much larger rod in the middle than is required for KL and $K'L'$ in the truss Fig. 18. The middle rod for the truss shown in Fig. 18 may be made very lig

Fig. 18. Pratt Truss. Inclined Ties. Truss-diagram

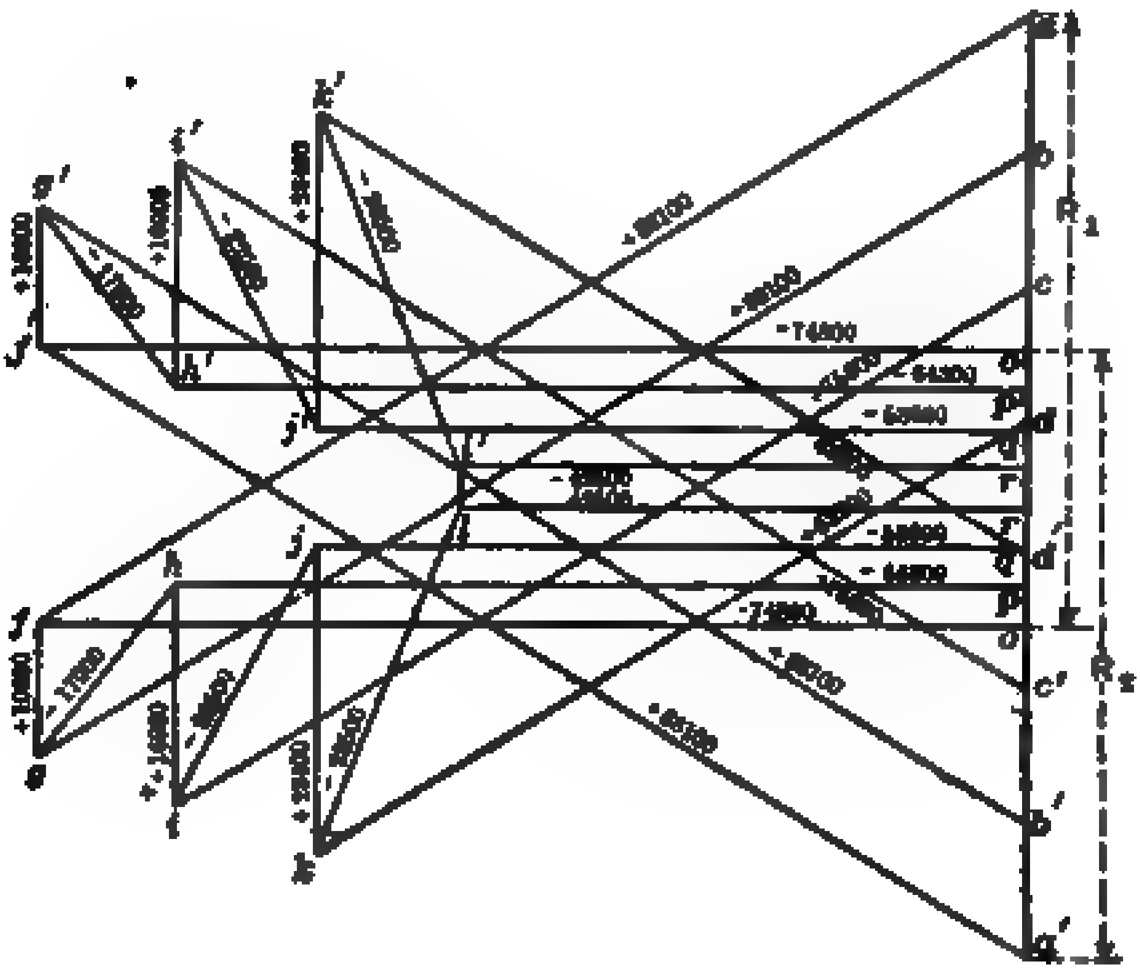


Fig. 18A. Pratt Truss. Inclined Ties. Stress-diagram.

This truss, however, requires, for good construction, special cast-iron wash for the rods.

Simple Fan Truss. **Example 9.** (Fig. 19.) This figure shows the skele of a simple FAN TRUSS with rafters inclined 30° and divided into three eq panels, making the loads AB, BC, CC', etc., equal. The stress-diagram drawn according to the principle already explained and requires no spe treatment. As the loads are equal, the stresses in this truss may be rea figured by means of Table X, and the student should compare the stresses t determined with those obtained by scaling the stress-diagram.

d Fink Truss. Example 1a. (Fig. 20.) The inclination of the ° and the distance between the trusses 20 ft. The loads are cal- a slate roof on boards or on angle-iron purlins. Commence the m by drawing a vertical line equal to the supporting force R_1, or d lettering the lower end of the line *o* and the upper end *a*, as these

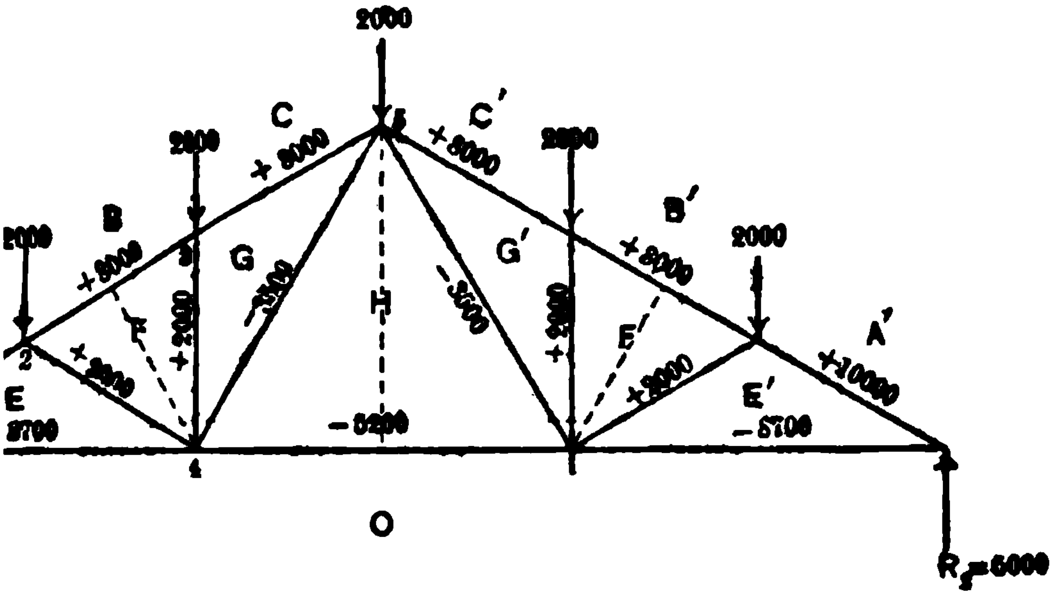


Fig. 19. Fan Truss. Truss-diagram

on each side of the supporting force at joint o. *an* and *no* are to *AN* and *NO*. For joint 1, *na* is drawn upward; *ab* is laid off ı lb and *bm* and *mn* are drawn parallel to *BM* and *MN*. At joint 2 : known, and *ml* is drawn parallel to *ML*, the sides of the stress- *on*, *nm*, *ml* and *lo*. At joint 3 a new condition is met, which is ay of the preced- and which is is form of truss, arently unknown a study of the however, it is and *IK* act as -RODS, taking up ı the lower ends t joints 2 and 5; s at joints 1 and *NM* and *IH* are h, the stress in as the stress in already known. :he number of at joint 3 to force known at the next *mb* and al to 16 100 lb.

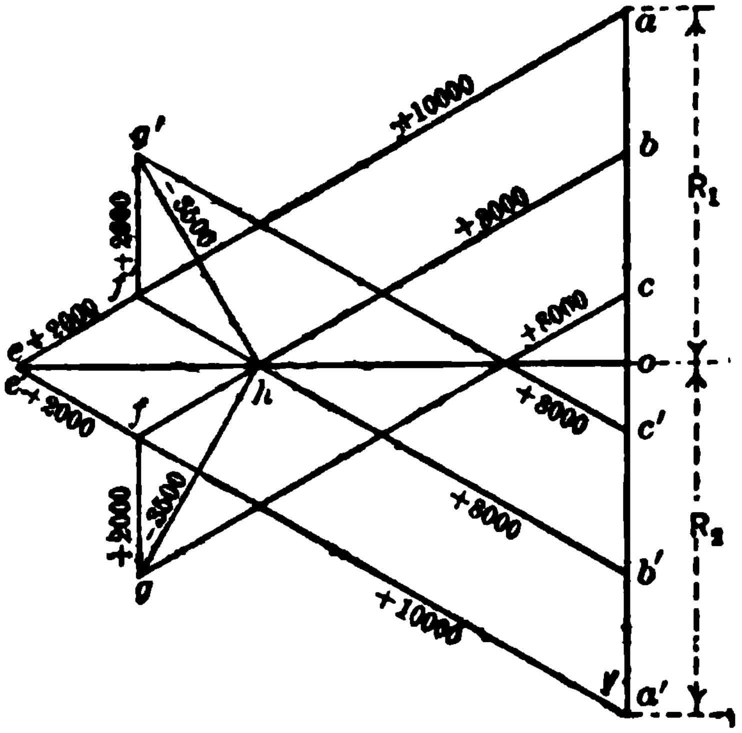


Fig. 19A. Fan Truss. Stress-diagram

s drawn parallel to *CI* and from *l*, the initial point, a line Between these two lines there must be a line, *ik*, parallel to ; length to *ml*; and this line is determined by means of the arallel ruler and straight-edge. If correctly drawn, the joint *i* ith *nm*. The sides of the stress-polygon for joint 3 are, then, and *kl*.

At joint 4 the stress-lines are *ol*, *lk*, *kg* and *go*.
At joint 5 the stress-lines are *gk*, *ki*, *ih* and *hg*.
At joint 6 the stress-lines are *hi*, *ic*, *cd* and *dh*.

If the stress-diagram is accurately drawn, a line from *d* parallel to the ra will pass through the point *h*. The vertical tie *GG'* (Fig. 20) has no st and its only purpose is to prevent the horizontal tie from sagging.

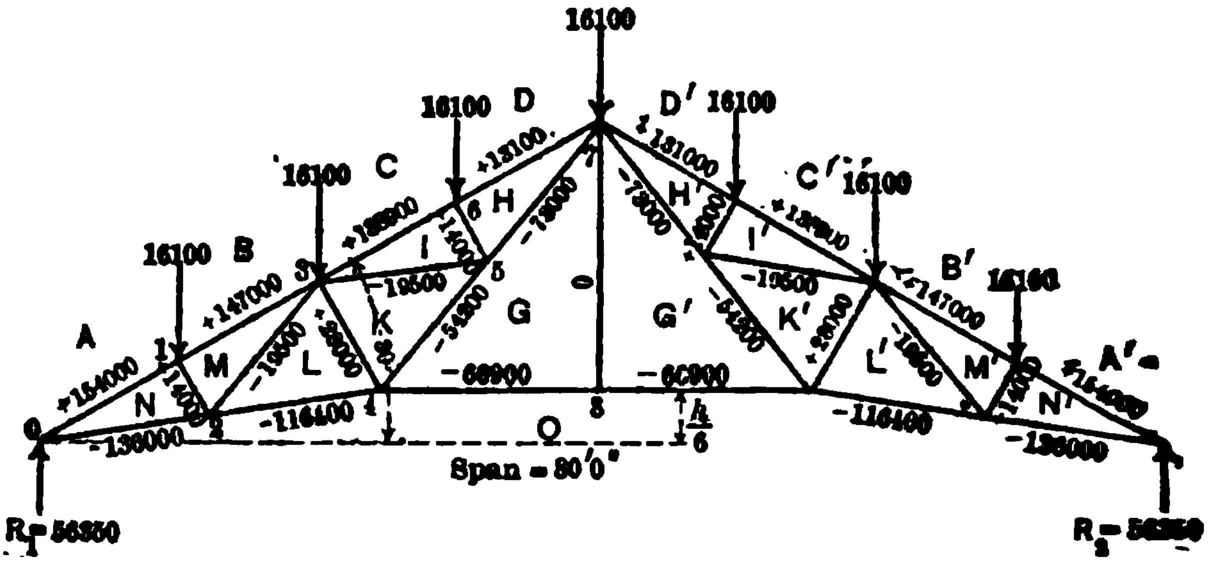


Fig. 20. Cambered Fink Truss. Truss-diagram

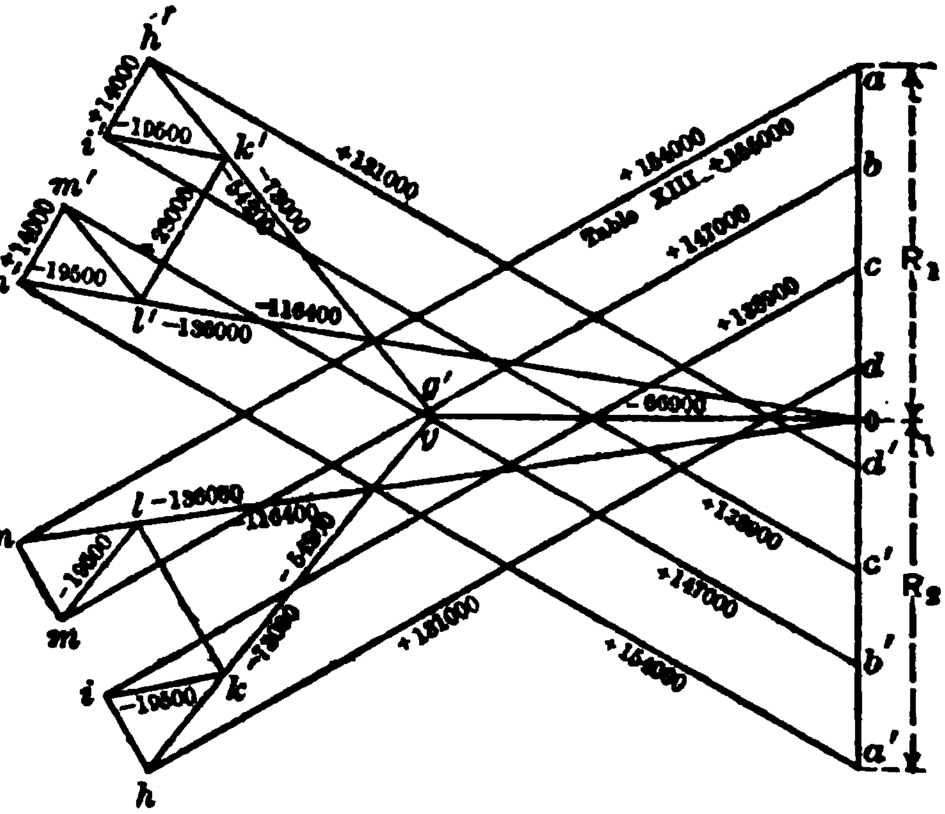


Fig. 20A. Cambered Fink Truss. Stress-diagram

Cambered Fink Truss. Example 11. (Fig. 21.) This is the truss sh in Fig. 20, with two additional loads. Steel trusses of this shape are required to support loads from below. In Fig. 21 there are two loads of 4 each, supported at joints 5 and 9, in addition to the roof-loads. The st diagram is drawn in exactly the same way as in Fig. 20A, except that at jo the first-known force *ro*, parallel to *RO*, 4 tons, is laid off, locating *r*. At joint, then, *ro*, *ol* and *lk* are known and *kg* and *gr* are drawn to close the poly It should be noticed that the stresses in *NM*, *IH*, *ML*, *KI* and *LK* not affected by the ceiling-load. This is evident by comparing Fig. 21A Fig. 20A. All of the other stresses, however, are increased becaus the increase in the supporting forces, the greatest increase being in *KG* and

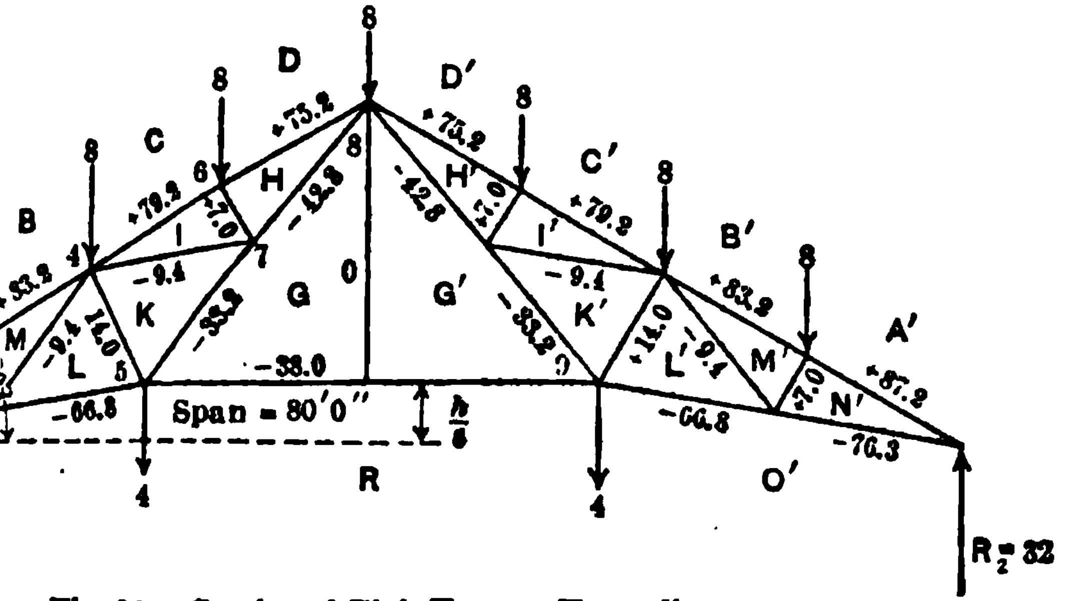


Fig. 21. Cambered Fink Truss. Truss-diagram

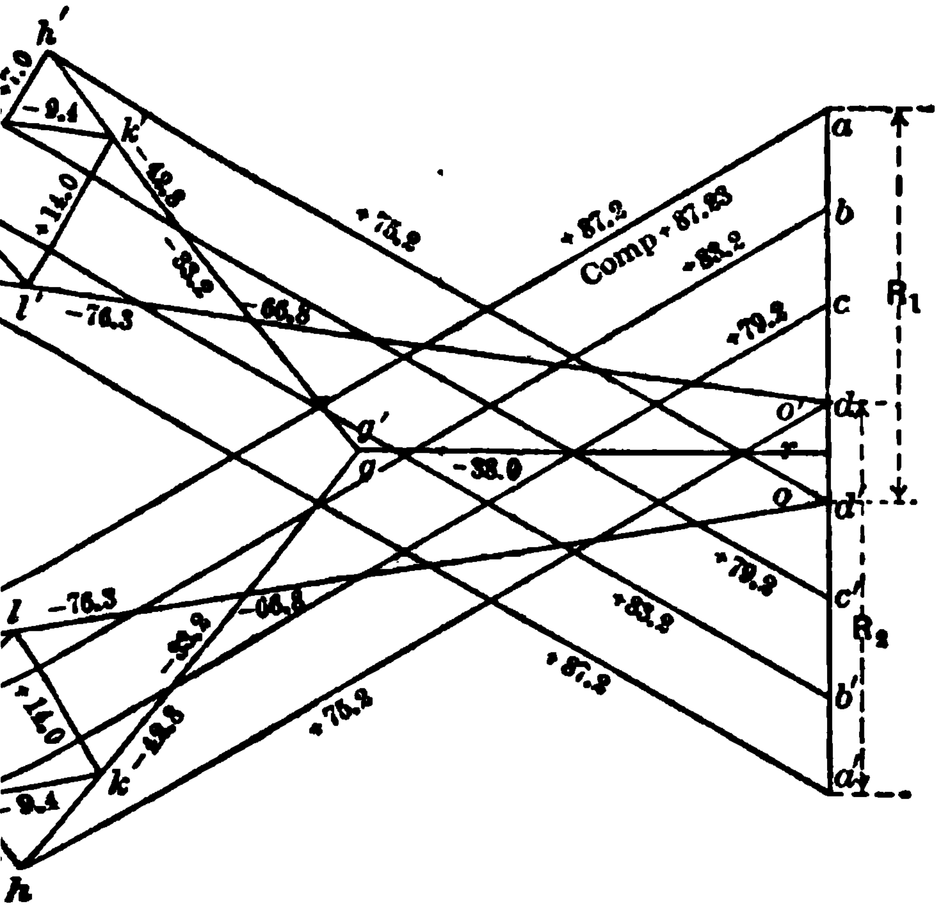
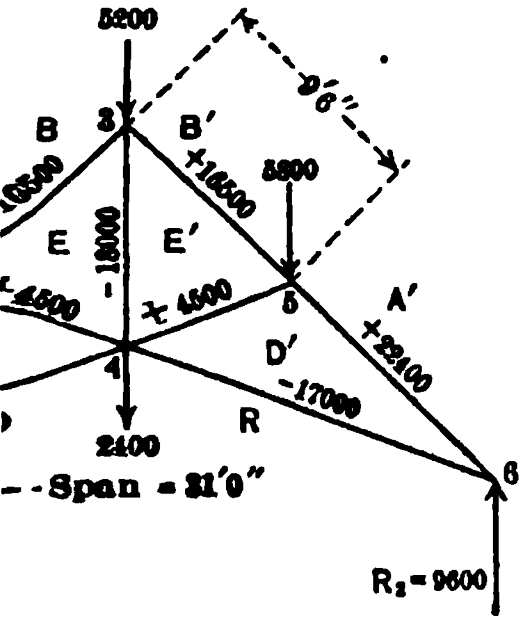


Fig. 21A. Cambered Fink Truss. Stress-diagram



...ors Truss. Truss-diagram

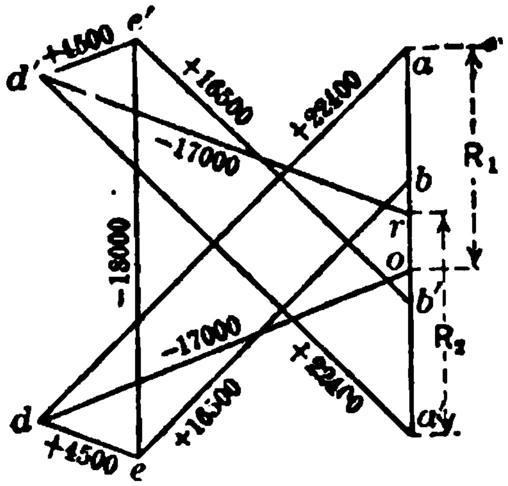


Fig. 22A. Scissors Truss. Stress-diagram

Simple Scissors Truss. Example 12. (Fig. 22.) This is the tru diagram of the truss shown in Fig. 22, Chapter XXVI, which is the simplest fo of the SCISSORS TRUSS. The truss-diagram is drawn by commencing with t line *oa* equal to the supporting force, 9 600 lb, and then in order the forces *bb'*, *b'a'*, *a'r* and *ro*, forming the polygon of the external forces. At joint 1, is known and *ad* and *do* are drawn, closing the polygon. At joint 2, *da* and are known and *be* and *ed* are drawn, closing the polygon. At joint 3, *eb* and are known and *b'e'* and *e'e* are drawn. This determines the stresses in one-h the truss. Those for the other half are, of course, of the same magnitude a character, but the stress-diagram should be continued for the second half of t truss as a check.

Scissors Truss. Example 13. Fig. 23 is the truss-diagram of the tru shown in Fig. 23, Chapter XXVI, with the loads figured about as they wou

Fig. 23. Scissors Truss. Truss-diagram

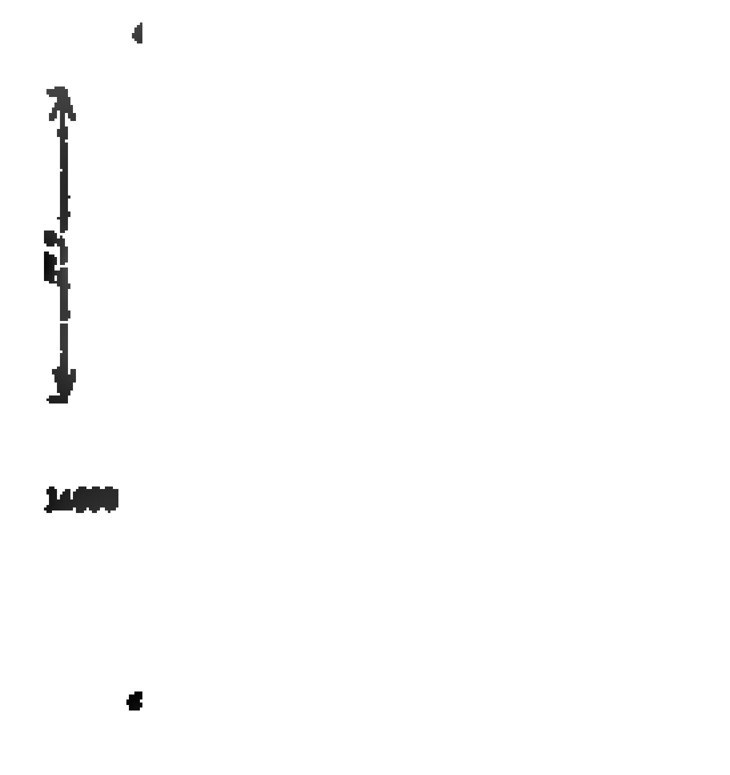


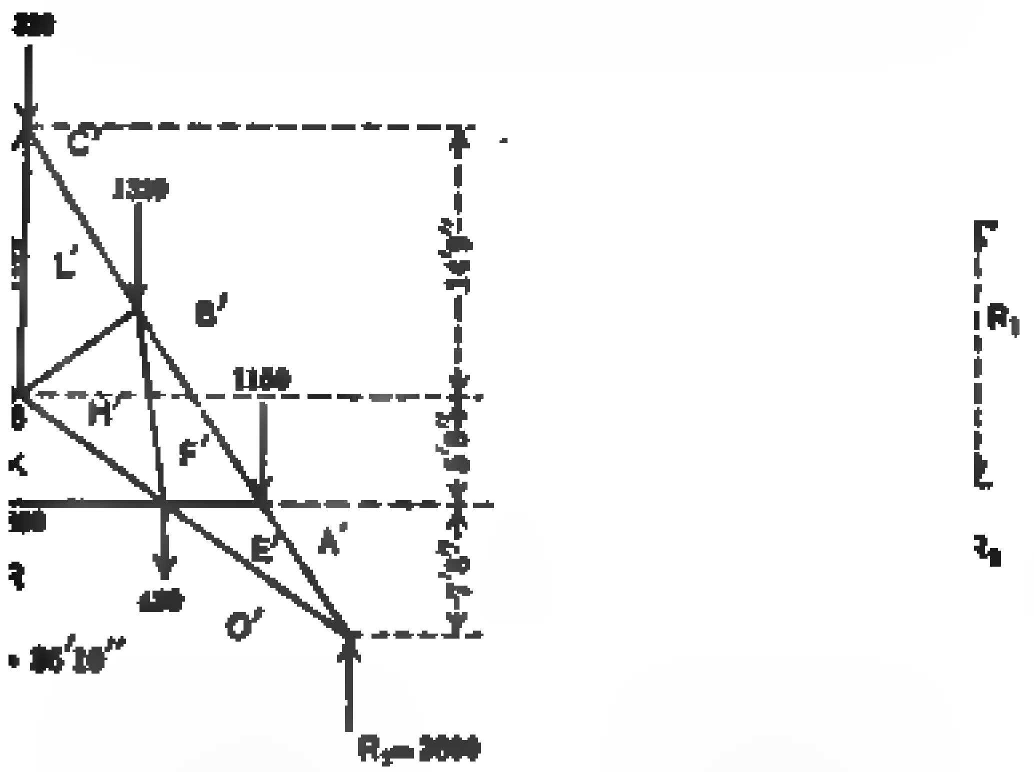
Fig. 23A. Scissors Truss. Str diagram

be for a slate roof and wooden ceiling and for a spacing of 12 ft on centers. T stress-diagram is begun by drawing the line *oa* equal to the supporting fo at joint 1 (14 600 lb). The sides of the stress-polygons for the different joi are as follows.

At joint 1: *oa*, *ae* and *eo*;
At joint 2: *ea*, *ab*, *bf* and *fe*;
At joint 3: *oe*, *ef*, *fh* and *ho*;
At joint 4: *hf*, *fb*, *bk* and *kh*;
At joint 5: *ro* (1 100 lb), *oh*, *hk*, *kl* and *lr*;
At joint 6: *lk*, *kb*, *bc* (5 400 lb), *cm* and *ml*;
At joint 7: *mc*, *cc'* (5 400 lb), *c'm'* and *m'm*.

The student should notice how much the stresses in the principal member this truss exceed the supporting forces or loads, and particularly the great st in the middle rod. For these reasons this is not an economical type of trus spans exceeding 36 ft.

Example 14. Fig. 24 is the truss-diagram of the truss
e 1056, and for which the roof and ceiling-loads are computed
1055. The truss shown in Fig. 4 is built of planks spiked
, but the stresses are found in precisely the same way if the
ivy timbers and supports a greater roof-area. It should be
ly the shape of the truss and the loads, including their point
:t the stress-diagram. The stresses at joints 1 and 2 are
nencing with *oa*, equal to R_1. At each of joints 3 and 4,
three unknown forces. We cannot obtain the stresses at
cting at joint 3 have been determined. The known forces
, equal to 430 lb, and the stresses acting in *OE* and *EF*; and
those acting in *FH*, *HK* and *KR*. *HK* and *KR* are in ten-
old joint 3 from falling down and outwards. Either one,
e omitted, and the greater the stress is in one the less it is in
' way to complete the stress-polygon for joint 3 is to fix the



s. Truss-diagram. (See, also, Fig. 24A. Scissors Truss.
ıpter XXVIII, Fig. 3) Stress-diagram

ie unknown stresses arbitrarily. The most satisfactory
to make the stress in *HK* equal to that in *KR*. This is
first known force at joint 3 is the load represented by *ro*,
ined by measuring upwards from *o*, 430 lb; next, the lines
From *f* a line is drawn parallel to *FH* and from *r*, a line
e two lines must be connected by a third line parallel to
l be drawn so that its length is equal to *kr*, which can be
viders. Lettering the ends of this line *h* and *k* the sides
-polygon for joint 3 are *ro*, *oe*, *ef*, *fh*, *hk* and *kr*. Knowing
are but two unknown forces at joint 4, and these are readily
he stress-polygon for joint 5 are *lc*, *cc'*, *c'l'* and *l'l*. Com-
ram with that of Fig. 23A, it is seen that the stress in the
s in proportion to the loads for the truss shown in Fig. 24
ı in Fig. 23, this reduction being due to the horizontal tie
s built of planks, spiked or bolted together, the form of
is preferable to that shown in Fig. 23.

Example 15. Fig. 25 is the truss-diagram of the scissors
27 of Chapter XXVI. The line *EF* in Fig. 25 does not

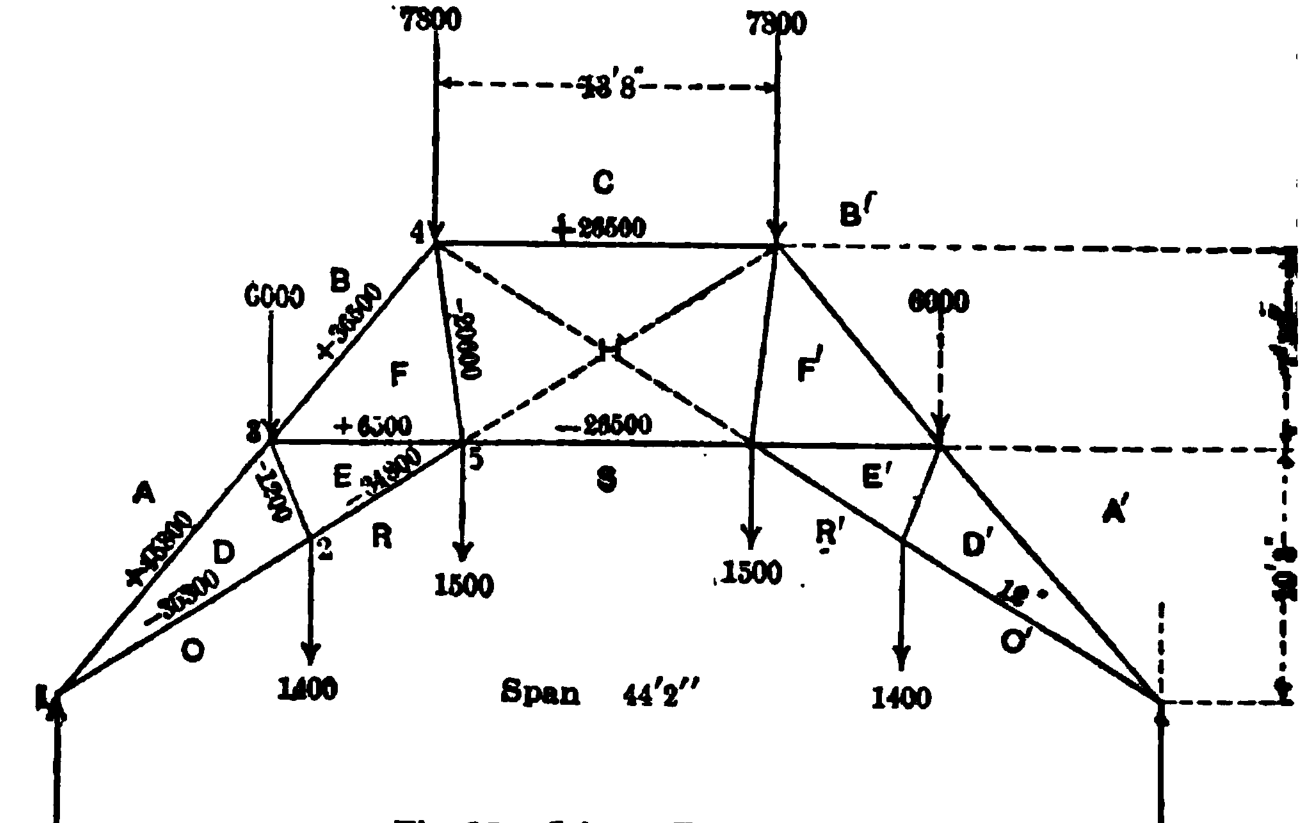


Fig. 25. Scissors Truss. Truss-diagram

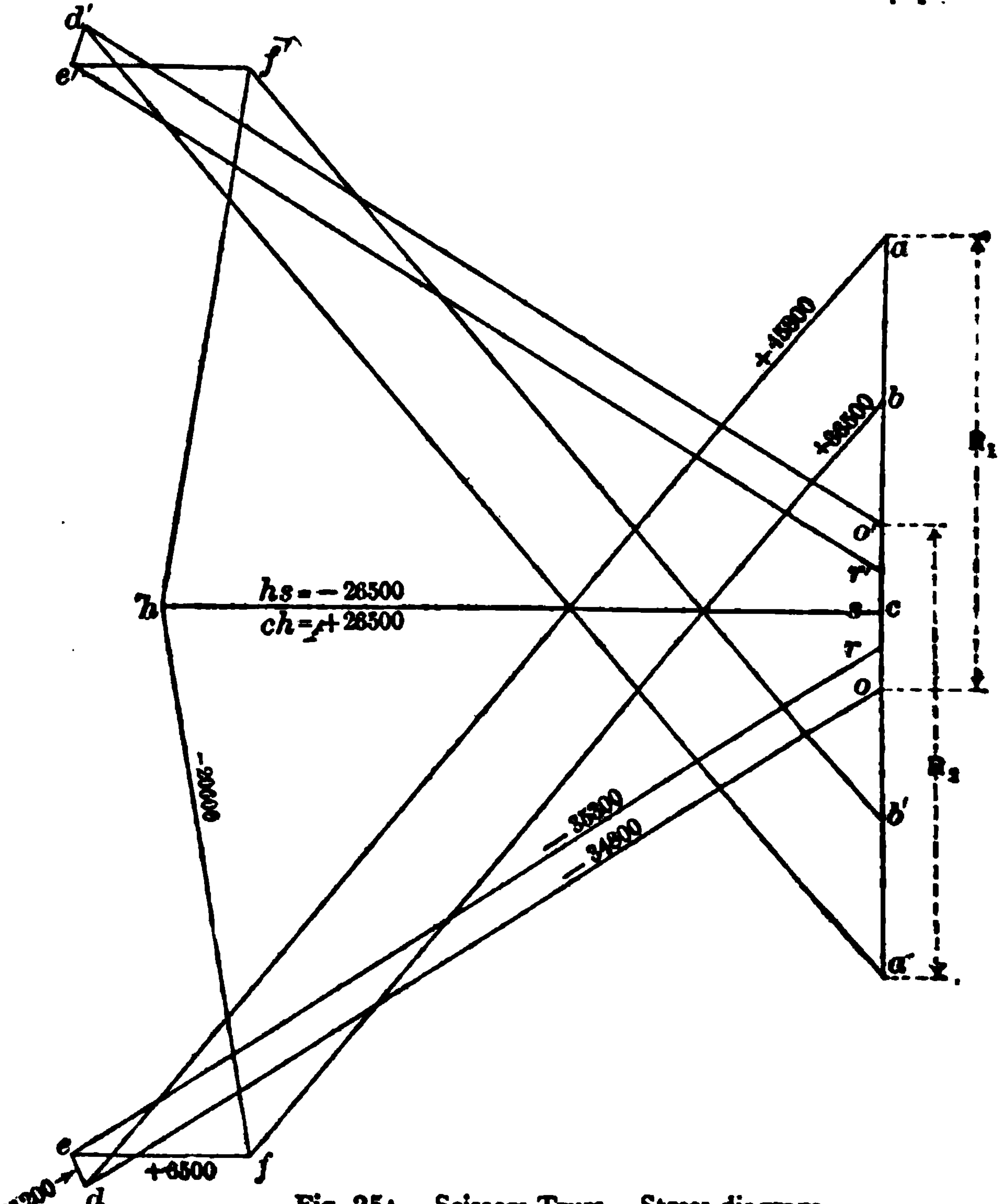
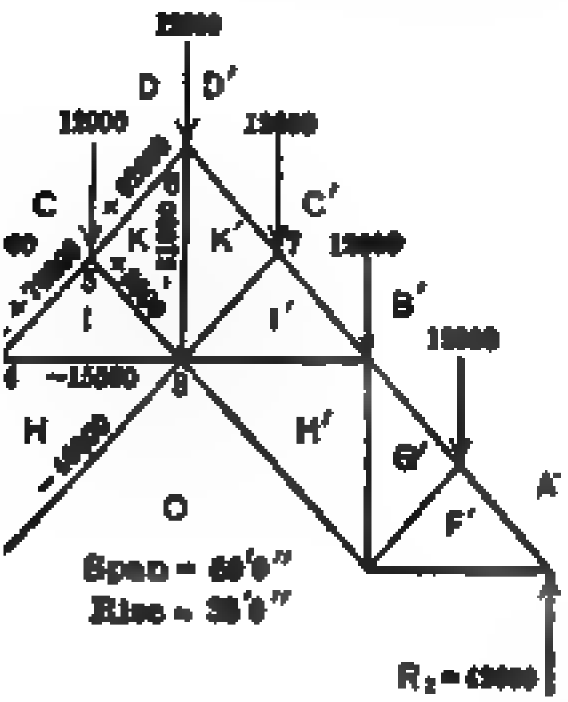


Fig. 25A. Scissors Truss. Stress-diagram

ith the center line of the strut in Fig. 27, because the inner end of
lropped slightly on account of the detail of the joint; but in truss-
lines must go from joint to joint, otherwise the stress-diagram can
. There are no stresses in the middle diagonals under a symmetrical
hence they are shown by dotted lines in Fig. 25. As no complica-
drawing the stress-diagram of this truss, a detailed description is
The sides of the stress-polygons for the different joints are as

For joint 1: *oa*, *ad*, *do*;
For joint 2: *ro*, *od*, *de*, *er*;
For joint 3: *ed*, *da*, *ab*, *bf*, *fe*;
For joint 4: *fb*, *bc*, *ch*, *hf*;
For joint 5: *sr*, *re*, *ef*, *fh*, *hs*.

ıcide, showing that the compression in *CH* is equal to the tension
plus and minus signs in Fig. 25, as in all the other diagrams,
ession and tension respectively.



uss without Tie-beam. Truss-diagram

Fig. 26A. Truss without Tie-beam. Stress-diagram

out Tie-Beam. Example 16. Fig 26 shows a truss which is
RS TRUSS nor a HAMMER-BEAM TRUSS, yet this form can be made
r to the HAMMER-BEAM TRUSS by inserting a curved brace below
lacing the pieces *OH* and *OH'* by curved members. There is no
wing the stress-diagram shown in Fig. 26A.

ıtal Thrust of Scissors Trusses. In the examples just given
med that the reactions are vertical and consequently that there
L THRUST. This would be true if the materials composing the
solutely rigid. This is not the case, however, and all trusses
geometrical lines of their shape change in shape after the full
In the SCISSORS TRUSS this changes the length of the span,
r and permitting the rafters to sag. If the trusses are con-
amber in the rafters and the span made a little short, the THRUST
orts can be practically eliminated by fastening one end of the
ing for a movement at the other, so that when the full roof and
e been placed on the truss the span will have its correct length.
is we must know HOW MUCH THE SPAN WILL CHANGE IN LENGTH
d. This can be determined in the manner shown in the follow-

ing example and by referring to Fig. 26B. Let Diagram 1 represent a simp SCISSORS TRUSS loaded as shown with 1 000 pounds at each top-chord joi and let the left end be assumed to rest upon rollers. Then both reactions w be vertical and the stresses in each member can be found from the usual stre diagram shown in Diagram 2. Let S be the stress in any member as fou from Diagram 2; u, the stress in any member produced by one pound acti horizontally at K and from L as found from Diagram 3; A, the area of a

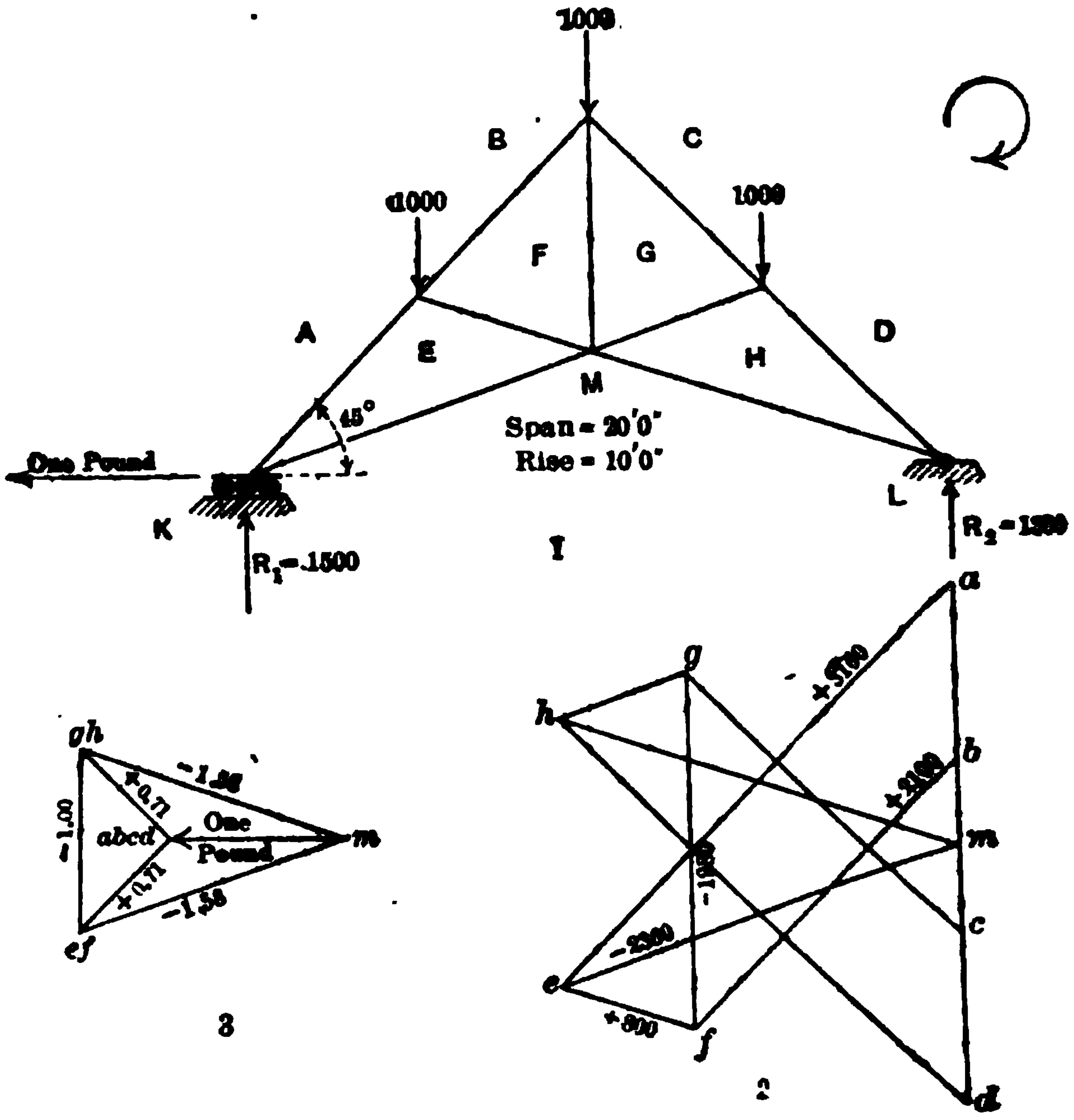


Fig. 26B. Simple Scissors Truss and Stress-diagrams

member, in square inches; l, the length of any member, in inches; E, You modulus of elasticity of the material composing any member and D, the to CHANGE IN LENGTH OF SPAN when the truss is subjected to its full load. Th

$$D = \sum \frac{Sul}{AE}^*$$

If H is the HORIZONTAL FORCE applied at K, which is necessary to make value of $D = 0$

$$H = D \div \sum \frac{u^2 l}{AE}$$

* Theory and Practice of Modern Framed Structures, Johnson, Bryan and Turn (John Wiley & Sons); Roofs and Bridges, Merriman and Jacoby (John Wiley & Son

l calculations for Fig. 26a are given in Table XVII, assuming that , excepting FG, are composed of 6 by 6-in white pine timbers with oo lb per sq in, and that FG is an upset round steel rod having 785 sq in with E equal to 30 000 000 * lb per sq in for steel.

II. Computations for D and H for a Particular Scissors Truss

(2) S, Diagram 2	(3) A	(4) $S+A$	(5) u, Diagram 3	(6) l	(7) $\frac{Sul}{AE}$	(8) $\frac{u^2l}{AE}$
+3160	36	87.8	+0.71	84.8	0.00528	0.00000118
+2100	36	58.3	+0.71	84.8	0.00351	0.00000118
+2100	36	58.3	+0.71	84.8	0.00351	0.00000118
+3160	36	87.8	+0.71	84.8	0.00528	0.00000118
−2360	36	65.5	−1.58	126.5	0.01316	0.00000875
−2360	36	65.5	−1.58	126.5	0.01316	0.00000875
+ 800	36	22.2	0	63.2	0.0	0.0
−1980	0.785	2522.0	−1.00	80.0	0.00672	0.00000340
+ 800	36	22.2	0	63.2	0.0	0.0
					0.05062	0.00002562

in and $H = 0.05062 + 0.00002562 = 1975$, or, approximately, is shows that the span would lengthen about 1/20 in, if allowed free one end; or, if fixed, there would be a HORIZONTAL FORCE of ng to push the supports out. In column 4 it is seen that the uare inch are only about one-tenth of those permissible. Assumads become 10 000 lb at each apex-joint, the HORIZONTAL DEFLEC- about 1/5 in, and the HORIZONTAL THRUST becomes 20 000 lb. nclusively that a large excess of material must be employed in the s, particularly in the members EM and HM which contribute over alue of D as shown in column 7, if the HORIZONTAL DEFLECTION is small that its effect may be neglected. As stated before, if the ted to deflect horizontally until fully loaded, the walls or supports bly no HORIZONTAL THRUST to resist.

ner-Beam Truss. As usually constructed the HAMMER-BEAM ted to exert more or less HORIZONTAL PRESSURE at the supports; vided for by heavy walls and buttresses. The diagram of such n in Fig. 27, in which the CURVED BRACES usually built in the the truss are not shown, as they are considered to be purely orna- vertical loading have no stresses. The brace OM is drawn as straight; but a curved brace may be used instead, without alter- n. The stress in the curved piece is that found from the stress- sed by the bending stress due to its curvature. To determine nembers of this truss it is necessary to first find the HORIZONTAL truss against the wall. To do this all the truss-members from 4 are considered to form a FRAMED BRACE, OF ASSEMBLAGE OF ng the upper portion of the truss at joint 4, or a SINGLE BRACE, oken line o4, Fig. 27, is assumed to have the same effect on the pieces put together in the FRAMED STRUT; that is, the truss is

lb per sq in is used for the value of E for steel the values of D and changed. See Table I, page 664.

considered to have the same HORIZONTAL THRUST as the truss shown in Fig. 26 The load at joint 4 is evidently: 12 000 lb, plus the load at joint 5, plus h the load at joint 6, plus half the load at joint 2; making in all, 36 000 lb. T find the HORIZO TAL THRUST and t stresses the proc dure is as follow ab (Fig. 27B) is la off equal to the lo at joint 2, bc equal the load at joint cd equal to the lo at joint 5, and d equal to the load joint 6. Then t load at joint 4 (Fi 27A) = ½ ab + bc cd + ½ dd'; and if horizontal line drawn from x to t left, and from t center of ab a li parallel to the li 4–0 (Fig. 27A) the

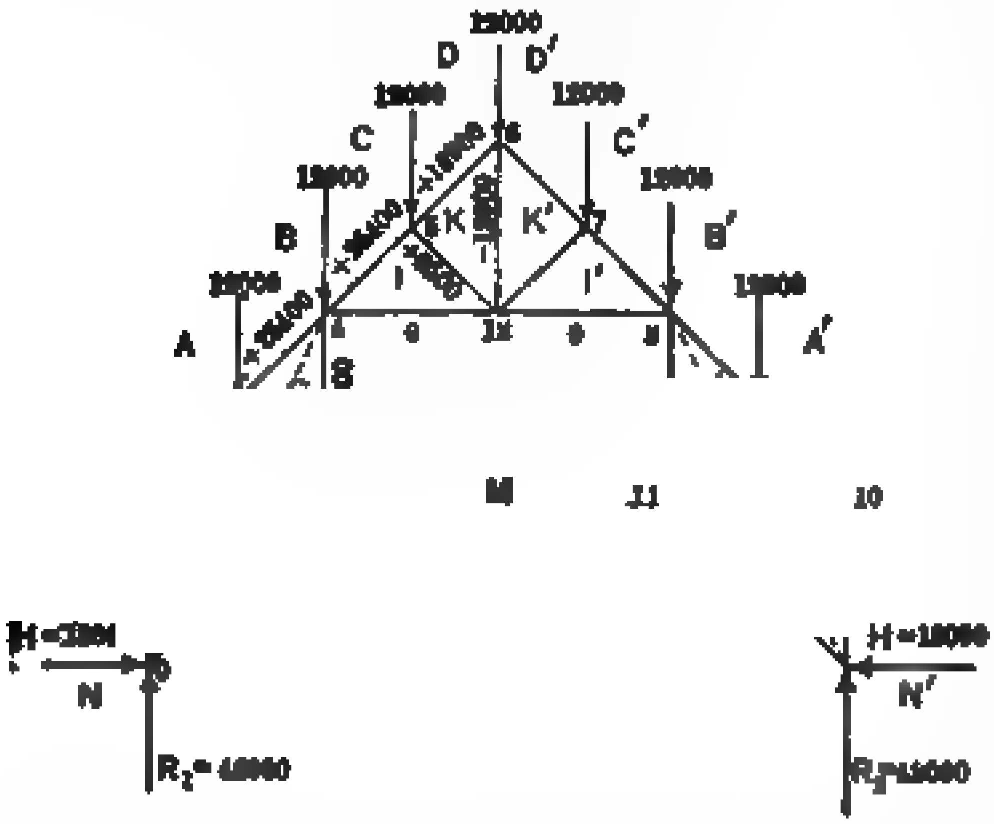


Fig. 27. Hammer-beam Truss. Truss-diagram

two lines will intersect at m, and mx is the MAGNITUDE OF THE HORIZONTA THRUST exerted on the wall at the joint o. Having obtained this thrust, is easy to determine the stresses in the pieces. At joint o the four forces

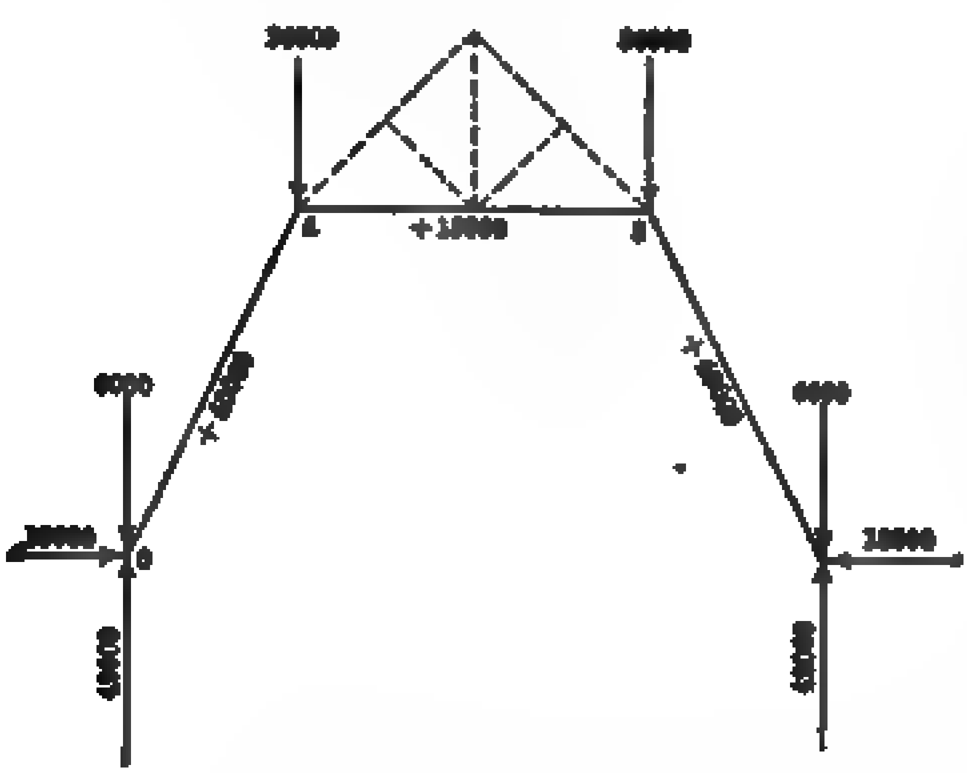


Fig. 27A. Hammer-beam Truss. Truss-diagram

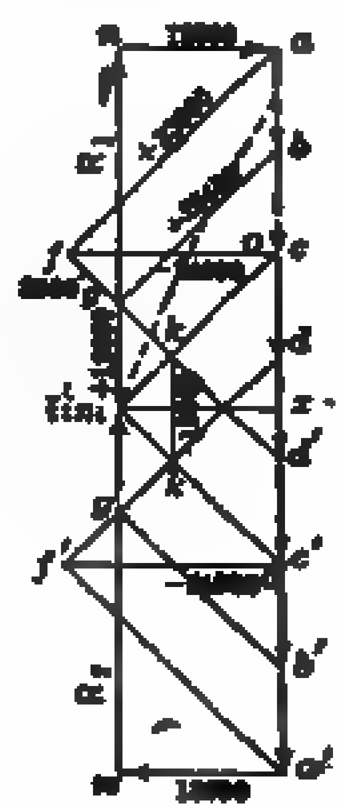


Fig. 27B. Hammer-be Truss. Stress-diagra

equilibrium are the resistance to the thrust, mx, the vertical supporting fo mn and the stresses ao and om, closing the polygon. At joint 1, oa, af and fo a the stresses in OA, AF and FO. At joint 3 the stresses are mo, of, fg and g at joint 2 they are fa, ab, bg and gf; at joint 4 the stresses are mg, gb, bc and

olygon. It will be noticed that the POLYGON CLOSES without allow- to be drawn parallel to *IM*; hence there is no stress in *IM*, with ing. When there are wind-loads there is some compression in *IM*, ber is a necessary part of the truss. At joint 5 there are the stresses *ki*, and at joint 6. *kd*, *dd'*, *d'k'* and *k'k*, which complete the stresses for russ, which are all that are needed. Comparing, now, the diagram

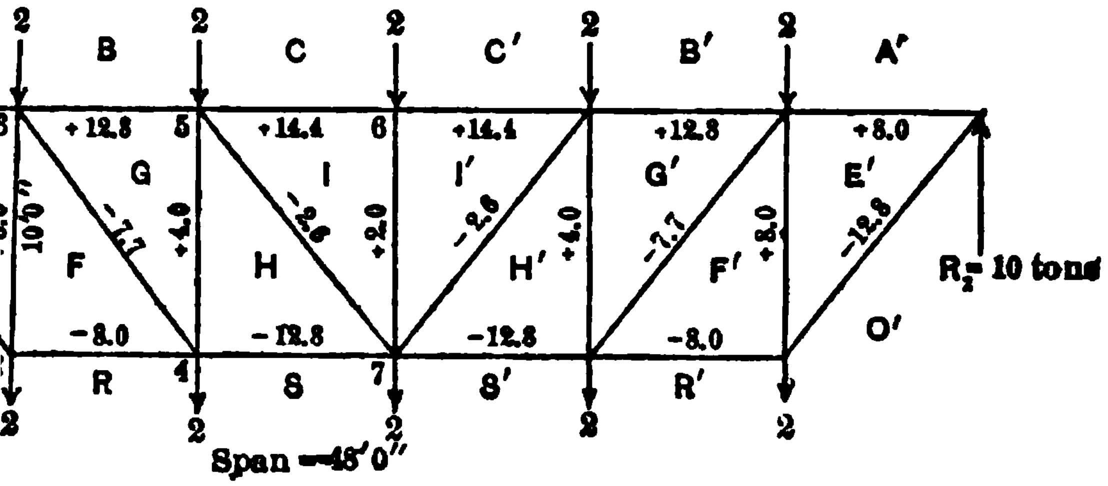


Fig. 28. Suspended Pratt Truss. Truss-diagram

. 27B, with Fig. 26A, it is seen that, in general, the MAGNITUDES SES in the truss, Fig. 27, are much less than those of the stresses ig. 26; while, on the other hand, the latter truss may be so con- t to exert the OUTWARD THRUST on the walls, exerted by the truss 27. If curved members are introduced between joints 3 and 12, they should be lightly secured at the ends or the stresses deter- e manner e SCISSORS

ed Pratt nple 17. be the of a SUS- T TRUSS, ed at top ith 2 tons t. The is drawn same way E TRUSS, diagonals site direc-

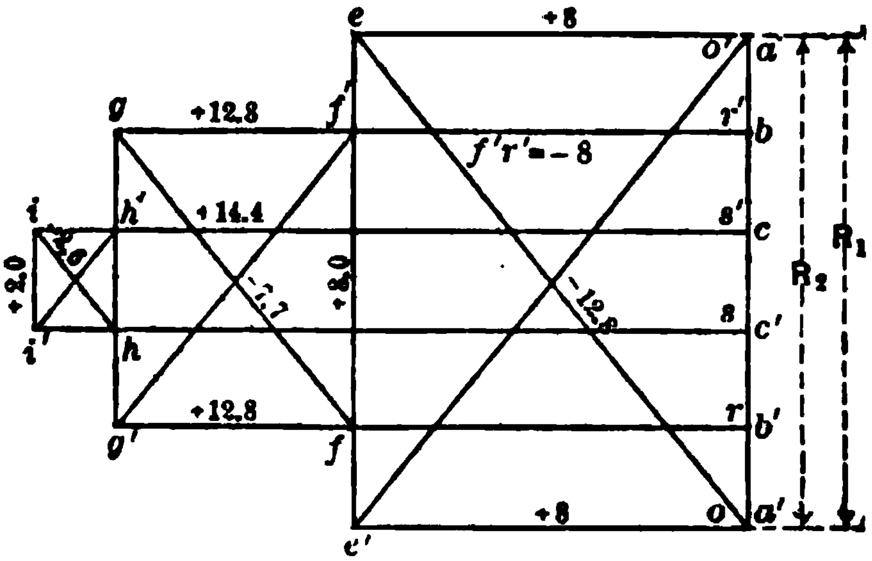


Fig. 28A. Suspended Pratt Truss. Stress-diagram

sses should be drawn for the joints in the order in which they In this truss the verticals are in compression and the diagonals

ss. Example 18. Fig. 29 is the truss-diagram of a light iron of 32-ft span, intended to support a tar-and-gravel roof, the right-angles to the line of the trusses. The stress-diagram is previous examples, taking the joints in the order in which they

ren Truss, or Lattice Truss. Example 19. The truss-dia- Fig. 30 is best analyzed by considering it as built up of two

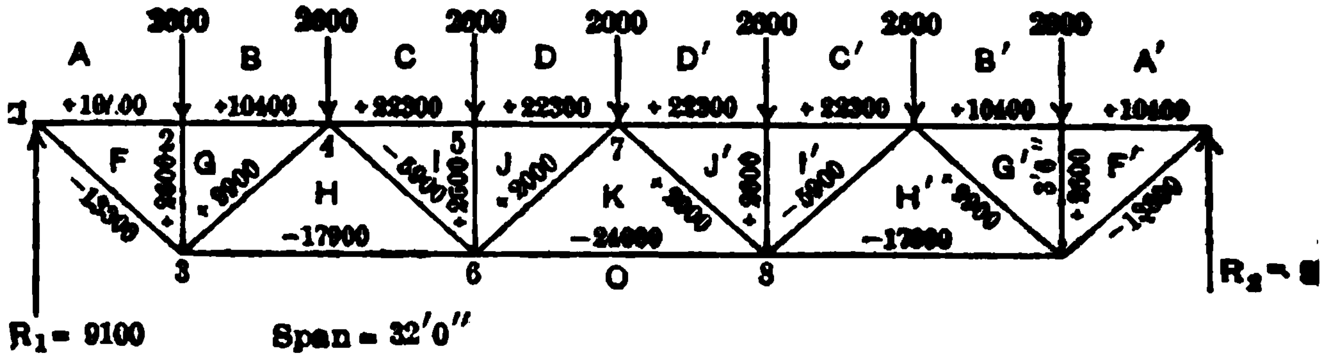


Fig. 29. Warren Truss. Truss-diagram

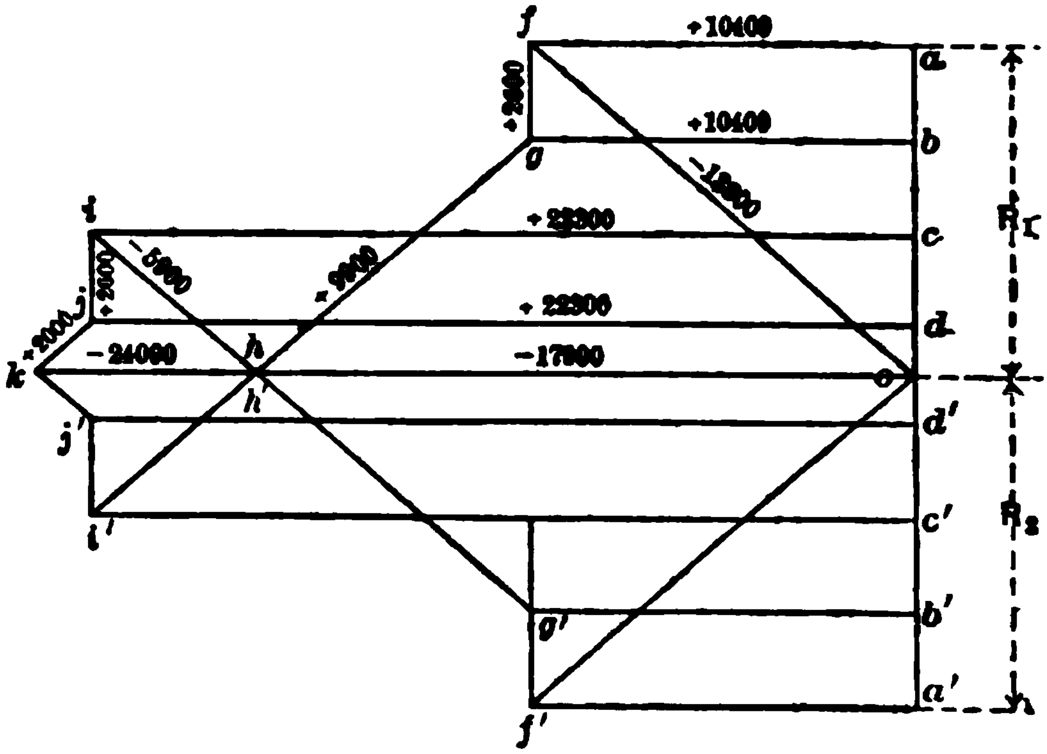


Fig. 29A. Warren Truss. Stress-diagram

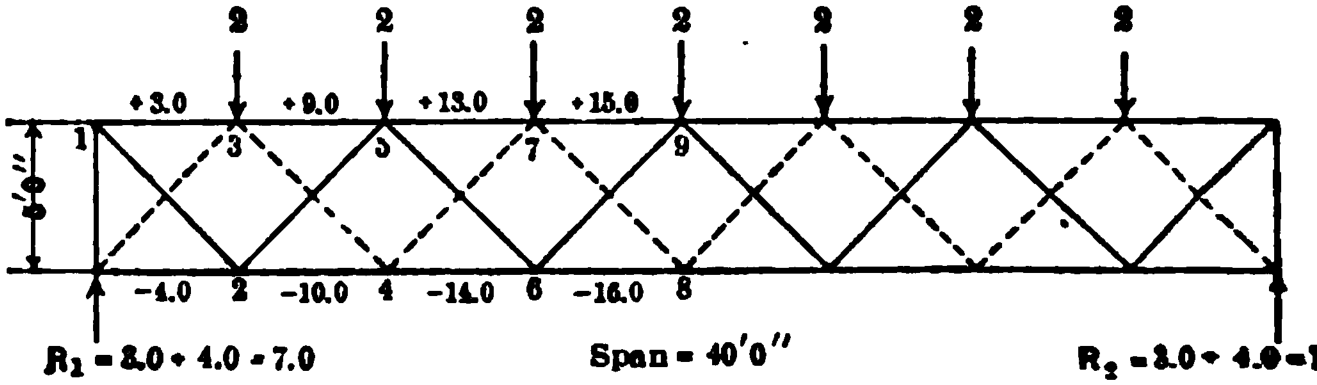


Fig. 30. Double Warren Truss. Truss-diagram

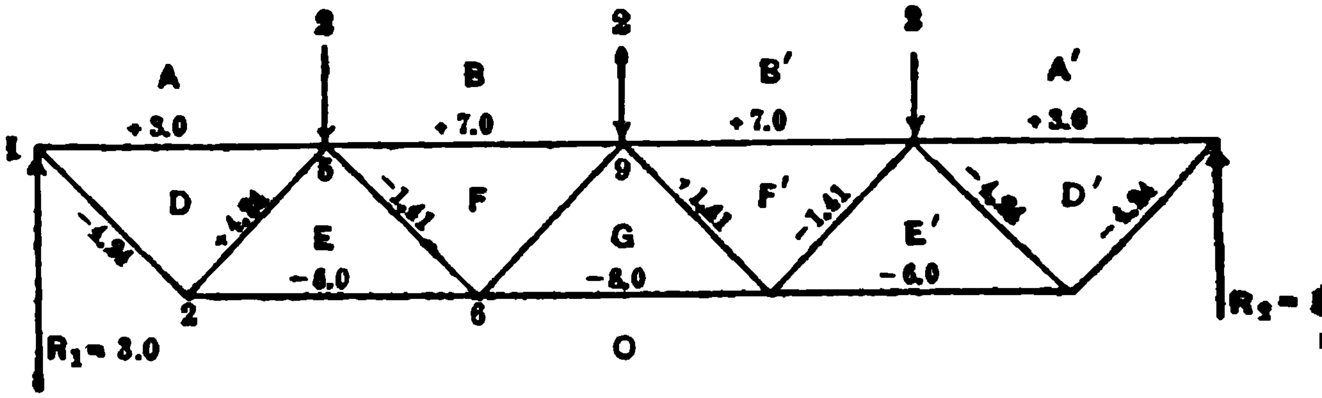


Fig. 31. Warren Truss. Truss-diagram

usses, laid one over the other, the full lines indicating a truss such n Fig. 31, and the dotted lines a truss as shown in Fig. 32. Three loads would come on the first truss and four on the second. The found for each truss separately nbined for the top and bottom is the stress in the top chord Fig. 30, would be that in AD, tons; from 3 to 5 it would be stress in AD, Fig. 31, plus that 32, or 9 tons; from 5 to 7 it ial to the stress in BF, Fig. 31, ss in BE, Fig. 32, or 13 tons, he stress in the bottom chord n the same way. The diagonal ies act independently of each

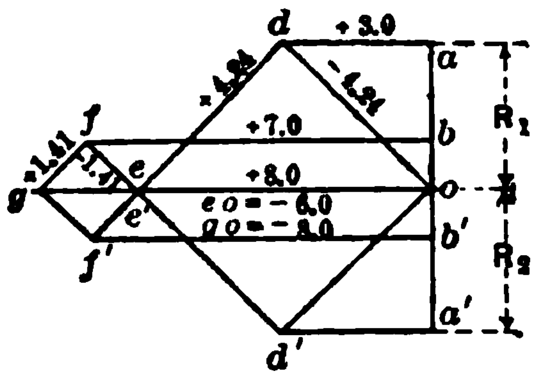


Fig. 31A. Warren Truss. Stress-diagram

e stresses are those indicated on the stress-diagrams. The plus ompression and the minus signs tension. In Fig. 32 the sides olygon for joint 7 are fe, eb, bc, and ge, which closes without a line parallel to GF, showing that there is no stress in the two

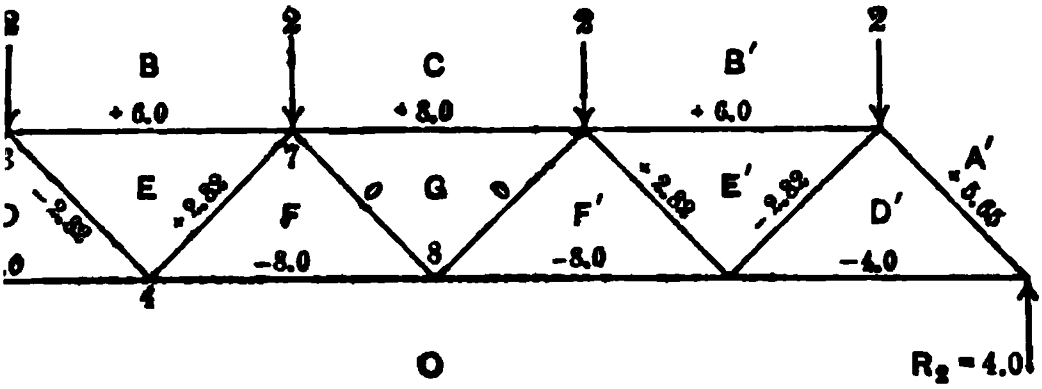
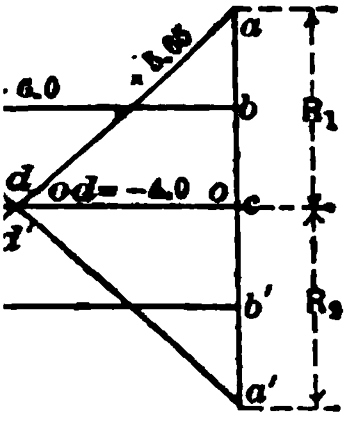


Fig. 32. Warren Truss. Truss-diagram

except that due to the weight of the bottom chord. This truss ructed of steel angles. When wood is employed, three or four ses are combined forming the LATTICE TRUSS shown in Fig.
It is entirely unnecessary to use graphical methods in determining the stresses, as the chords and web-members are respectively uniform in size. For the chords the maximum bending moment divided by the distance, center to center, of the chords gives the designing-stress for the chords. The maximum vertical shear, usually the reaction, divided by the number of simple WARREN TRUSSES combined gives the vertical component for which the web-planks are designed. This, of course, leads to a waste of material as far as resisting stresses is concerned, but for stiffness and economy in labor, the extra used. This truss can be extended indefinitely by giving it for the span. (See, also, pages 1008 and 1009.)



n Truss. Stress-diagram

r Truss. Example 20. Fig. 33 is the truss-diagram of a truss own in Fig. 59, Chapter XXVI, the panel-loads being taken at the analysis being the same for any other loads. The stress-

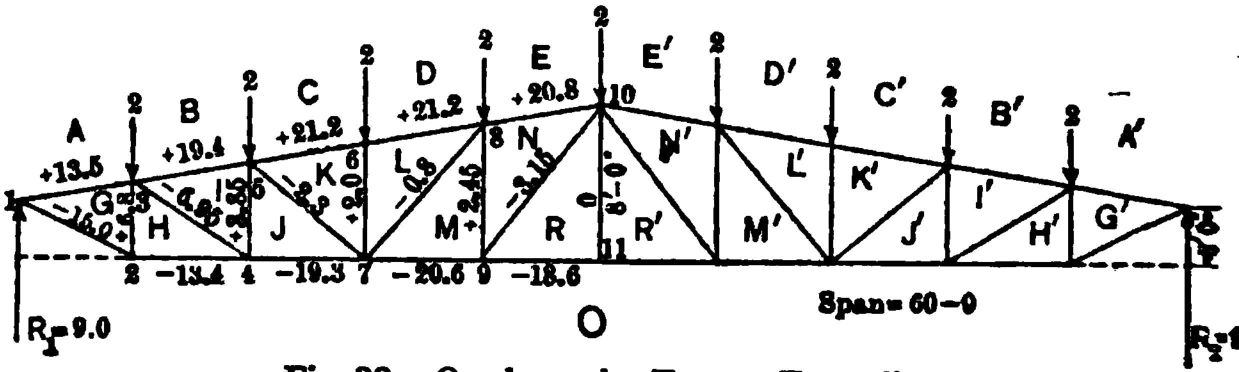


Fig. 33. Quadrangular Truss. Truss-diagram

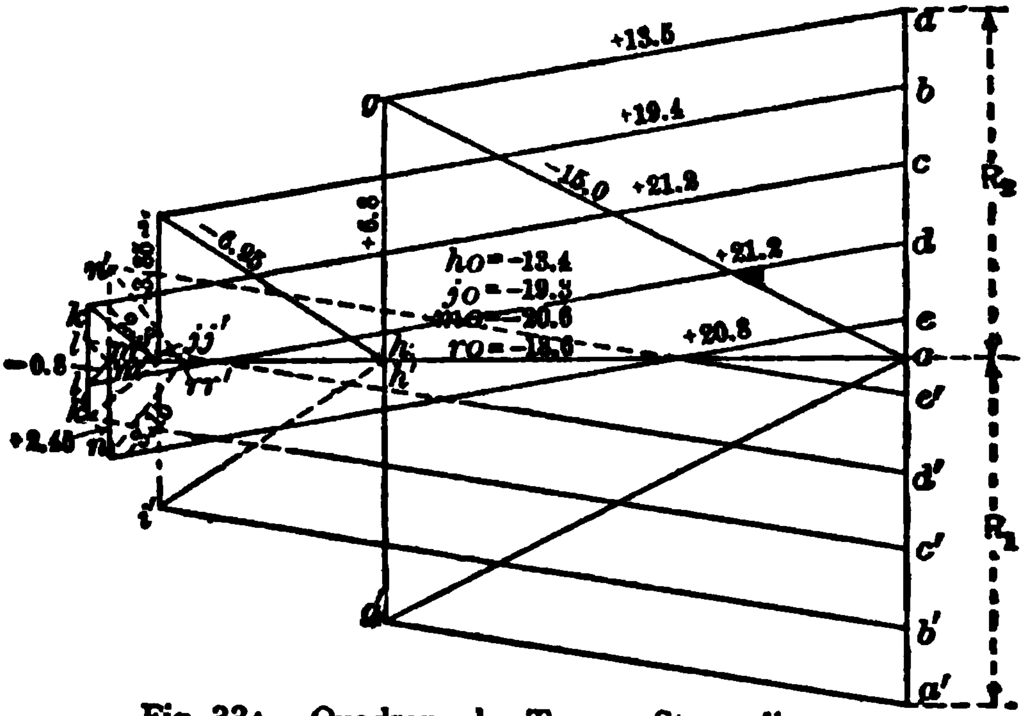


Fig. 33A. Quadrangular Truss. Stress-diagram

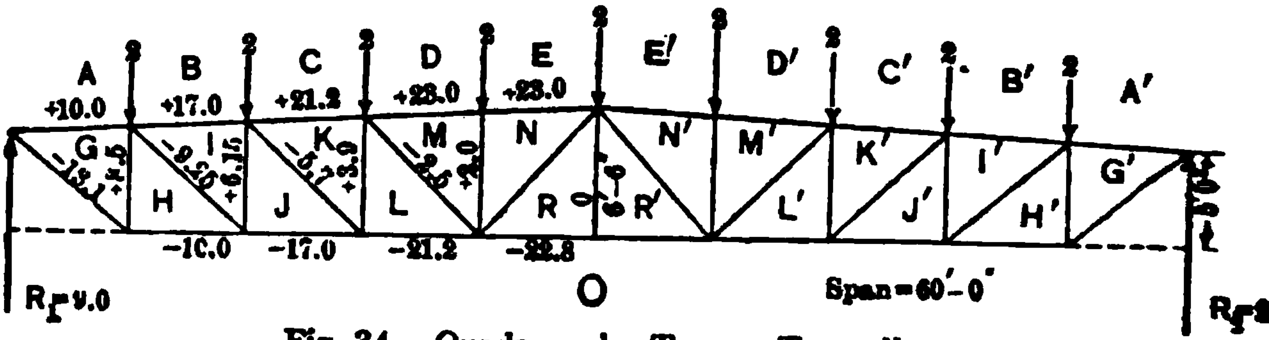


Fig. 34. Quadrangular Truss. Truss-diagram

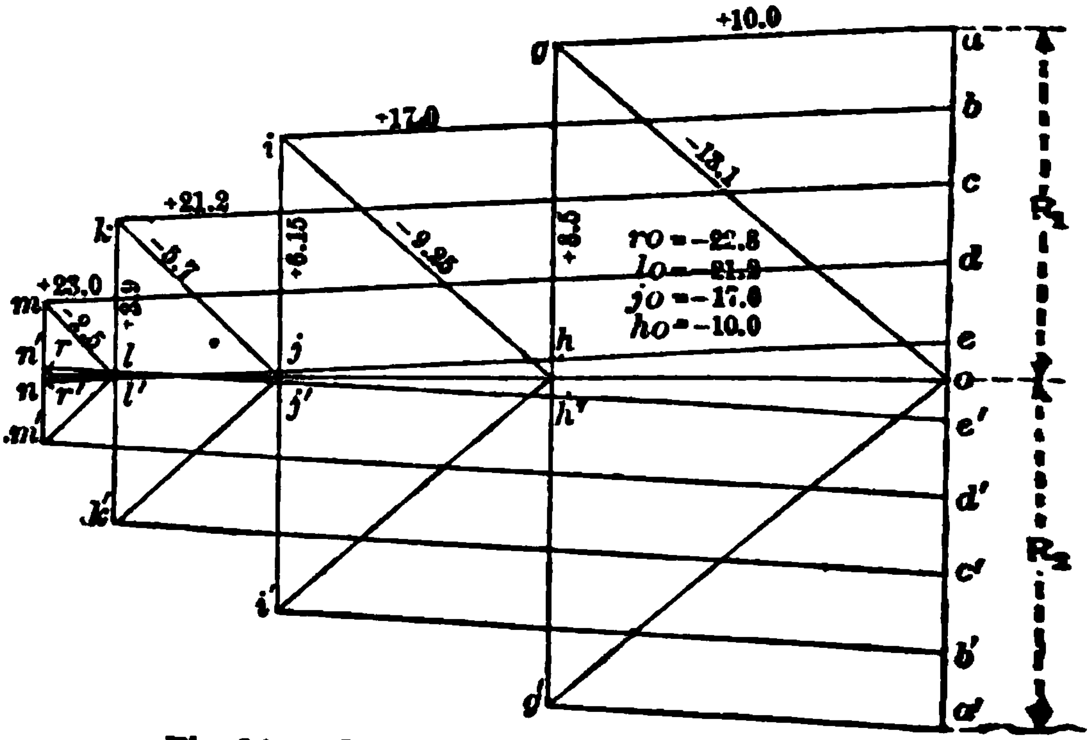


Fig. 34A. Quadrangular Truss. Stress-diagram

lrawn exactly as in the previous examples, commencing with the
orce *oa* and considering the joints in the order in which they are
In this truss the diagonal web-members are all in tension and the
ompression. It will be noticed that the inclination of the diagonals
nels nearest the middle of the truss is opposite to that of the diagonals
anels. This is due to the inclination of the top chord, which causes
stresses in the inner diagonals when they incline the other way.
LM, however, is so small that a single steel angle resists either a com-

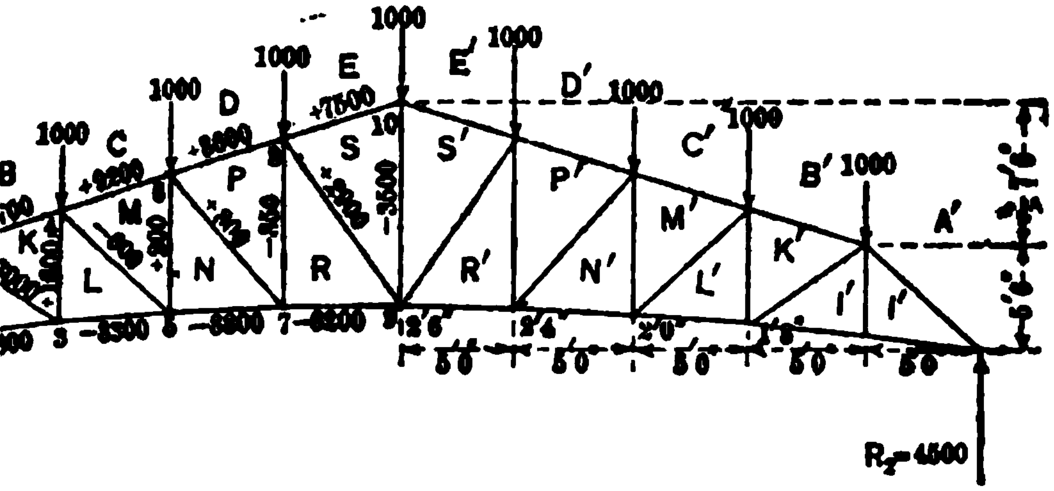


Fig. 35. Quadrangular Truss. Truss-diagram

sile stress. The truss shown in Fig. 34 is very similar to that shown
principal difference being that the slope of the top chord is less
than in the latter. In Fig. 34, the diagonals in the two middle
cline from the top of the middle vertical, and the stress in these
ery small. With a still less inclination to the top chord, the stress
zero; and with a horizontal top chord the character of the stress
sed. To keep it in tension, its direction should be changed, as in
oss, Fig. 28. Comparing the stresses in these two trusses, it is
le the stresses in the
less in Fig. 34A than in
tresses in the chords at
considerably greater.
less the height of a
rtion to the span, the
tresses in the chords,
e middle of the truss.
lar Truss. Example
is a truss-diagram
russ shown in Fig. 66,
I. The truss-diagram
he previous examples,
this case, as the
has different inclina-

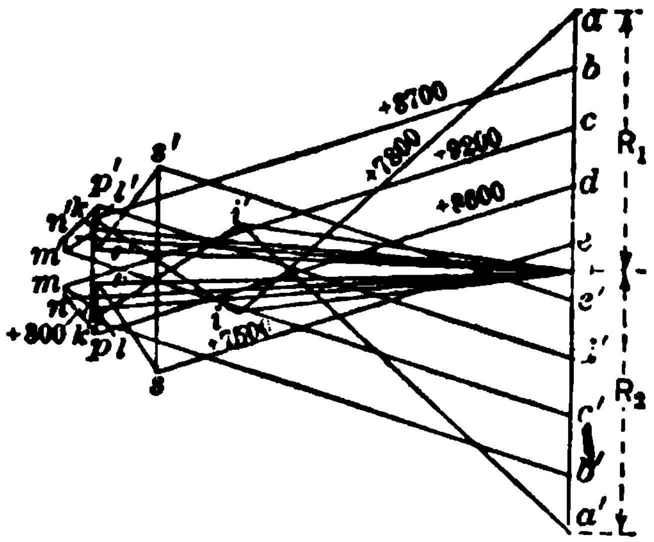


Fig. 35A. Quadrangular Truss. Stress-diagram

ferent panels, the stress-lines do not lie over each other, but
he lines in the stress-diagram being parallel to the corresponding
ss-diagram. In this truss the character of the stresses in the
embers is reversed in the two panels nearest the middle. Thus
stress-polygon for joint 4 are *lk*, *kb*, *bc*, *cm* and *ml*, the stress *ml*
joint and hence denoting tension. At joint 8 the sides of the
re *rp*, *pd*, *de*, *es* and *sr*, the latter line acting towards the joint,
ting compression. Under irregular loading, the character of
would probably be reversed, so that the piece would be in ten-

sion instead of in compression. The stresses in members of trusses like Figs. 3
34 and 35 should, therefore, always be computed for snow on one-half of t
truss only and also for wind-pressure.

Quadrangular Truss. Example 22. In Fig. 36 is shown the diagram
the truss illustrated in Fig. 65, Chapter XXVI. This truss is similar to th
shown in Fig. 34, except for the secondary bracing in the panels and for t

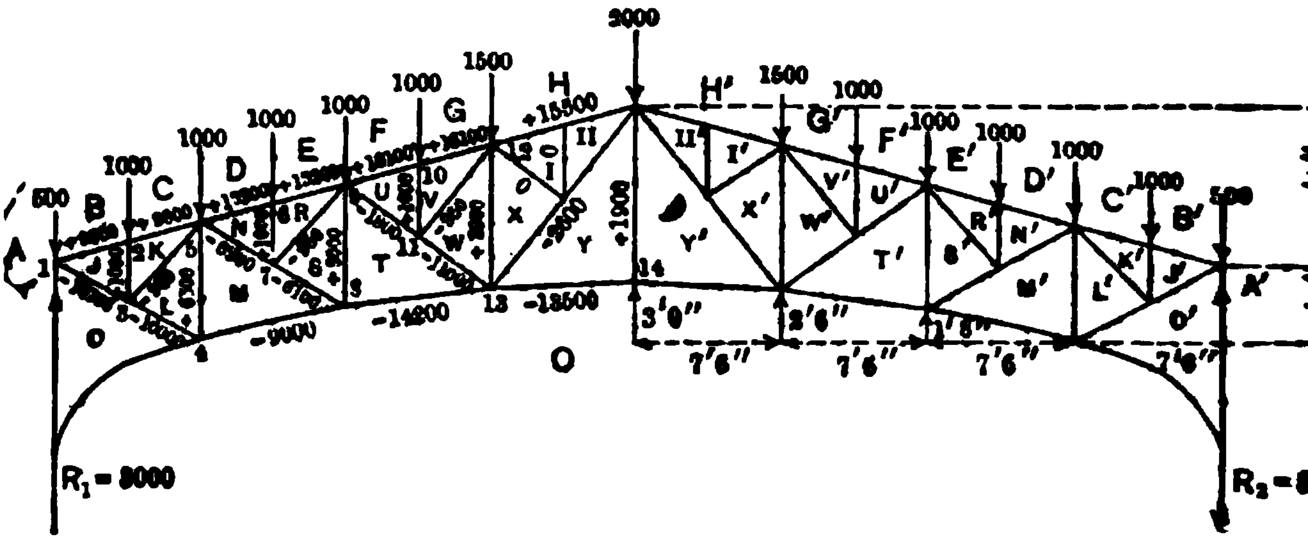


Fig. 36. Quadrangular Truss. Truss-diagram

curved bottom chord. The stress-diagram presents no difficulties. In drawi
the lines from *o* parallel to the members of the bottom chord the latter sho
be considered as made up of straight lines connecting the joints. Thus *os*
drawn parallel to an imaginary straight line connecting joints 8 and 4. As th
is no load over the center of the two panels next to the middle of the truss, th
are no stresses in the truss-members between *X* and *I* and *I* and *II*. Wh
the bottom chord is straig
as in Fig. 34, there is
stress in *YY'*; but when t
chord is curved, a tensio
stress develops, in *YY'*, t
magnitude of this stress be
indicated by *yy'* (Fig. 36
When the diagram is co
pleted for the entire truss,
is symmetrical about a ho
zontal line drawn through

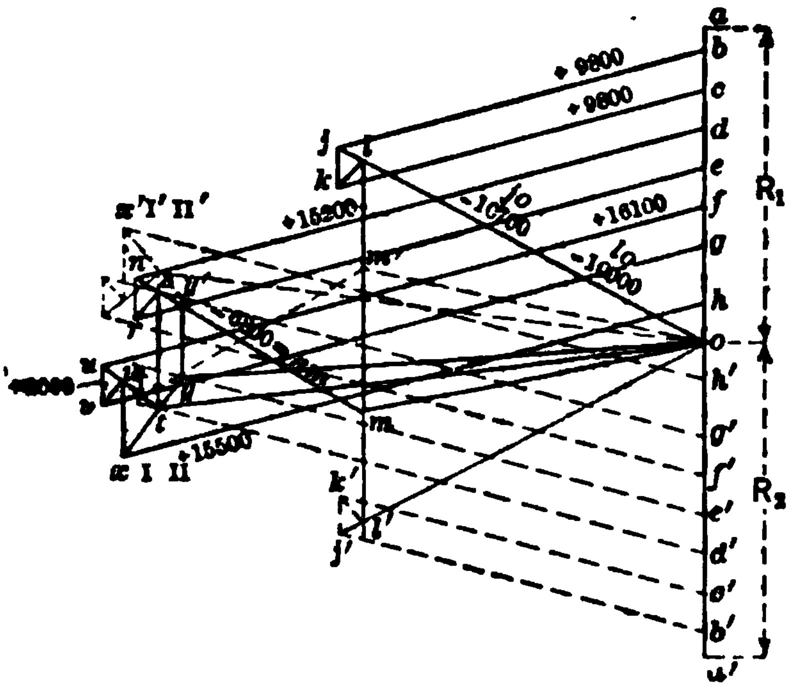


Fig. 36A. Quadrangular Truss. Stress-diagram

Bowstring Truss. Examp
23. The span of this truss
90 ft; the distance betwe
trusses from centers, 20
and the rise of the arch
rafter or upper chord, 20
The form of truss represent
in Fig. 37 is one of the m
economical for very great spans. In trusses similar to the one explained in t
example, the top chord is curved and is the only piece that is in compressi
All the other members are in tension. Under a steady load only, such as t
weight of the roof itself, the diagonals drawn with solid lines and placed
shown in Fig. 37 are all that are needed; but when there is a severe wind-pr
sure on one side of the roof only, it is necessary to have the additional set
diagonals shown by the dotted lines. These COUNTERBRACES, as they

lled, forming the additional set, are not stressed when there is a vertical load ly and they are omitted in drawing the stress-diagram. To draw the stress-agram, the loads are laid off on a vertical line, as in all the previous examples, e point *o* being half-way between *e* and *e'* (Fig. 37A). *oa* is the supporting ce at joint 1. In drawing the stresses at the different joints, those at joint 1 e first drawn and then those at joints 2, 3, 4, 5, etc., in the order in which they e numbered (Fig. 37). In the stress diagram, *oa*, equal to the supporting

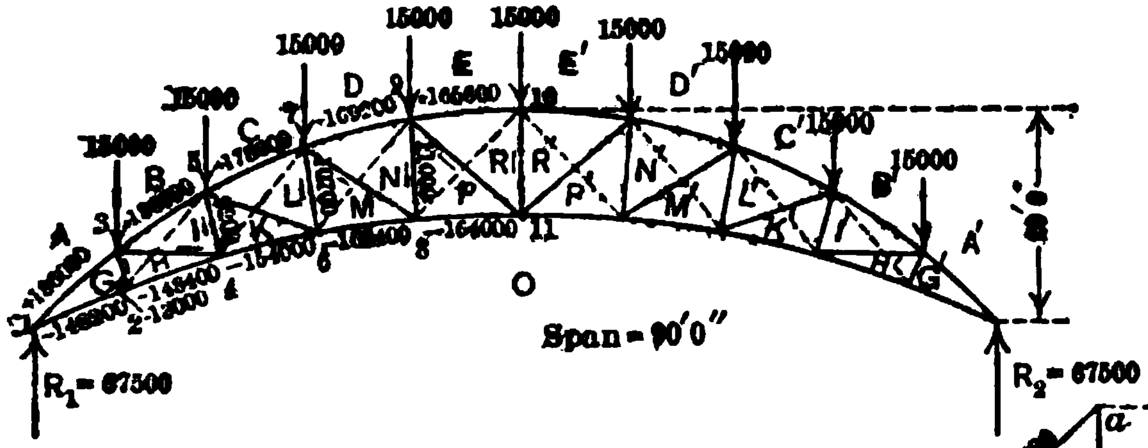


Fig. 37. Bowstring Truss. Truss-diagram

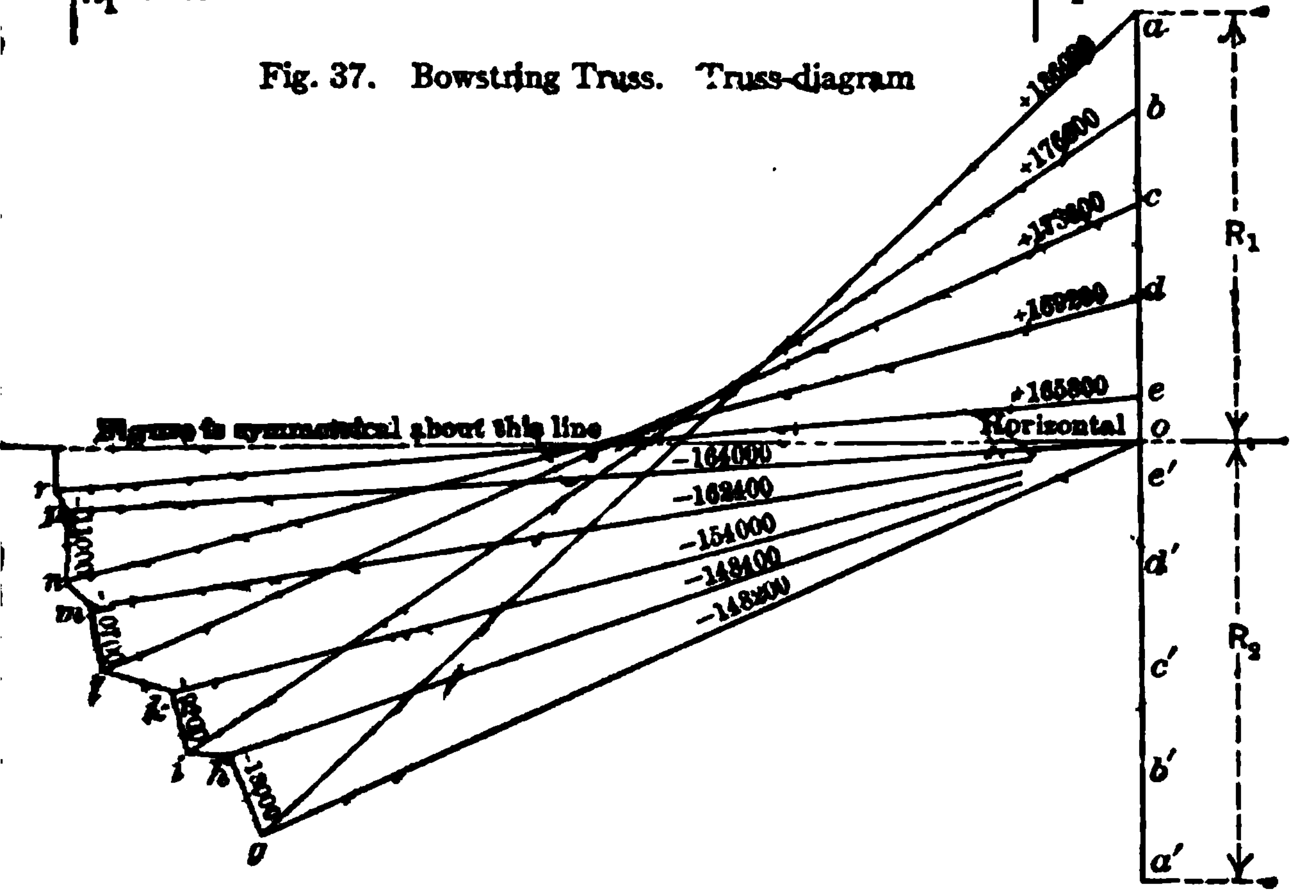


Fig. 37A. Bowstring Truss. Stress-diagram

e at joint 1, is known and from *a* a line is drawn parallel to *AG*, and from line parallel to *GO*. These two lines intersect at *g*. The lines repre-ing the stresses in the curved members of the truss are drawn parallel traight lines connecting the two ends of each curved piece. Thus *ag* is n parallel to 1–3 and *og* parallel to 1–2. At joint 2, *og* is known, *gh* is drawn llel to *GH* and *ho* parallel to *HO*. At joint 3, *hg* and *ga* have been drawn, ed *ab* is known and *bi* and *ih* are drawn. At joint 4, *oh* and *hi* have been n, and *ik* and *ko* are next drawn to close the polygon. The stress-lines for 6 and 8 are drawn in a similar way, and those for 5, 7 and 9 similarly to at joint 3. After drawing the stress-lines for joint 9, joint 10 is next con-d; and after the stress-lines for that joint are determined the stresses in e members of the truss are known. The stresses in this particular example iven in pounds on the respective lines in the stress-diagram. It will be

noticed that the stresses are very great in the top and bottom chords, but v small in the bracing. The latter stresses are, in fact, so small that it is jus well to make all the diagonal braces the same size and of dimensions suffici to resist the stress in IH, which has the greatest stress; or IH and KL may made the same size and MN and PR a smaller size. The verticals or radiat pieces may all have a sectional area sufficiently large to safely resist the st in NP. The great advantage of this truss lies in the fact that all its parts in tension excepting the upper chord, which, of course, is in compression. T manner in which stresses act may be described in general by saying that upper chord carries all the load, like an ARCH, and is prevented from spread out at the ends by the lower tie; and that the object of the bracing and vertical pieces is only to keep the bottom chord in its curved position.

Trusses Unsymmetrically Loaded. Now that the principles have b explained according to which the stress-diagrams may be drawn for seve

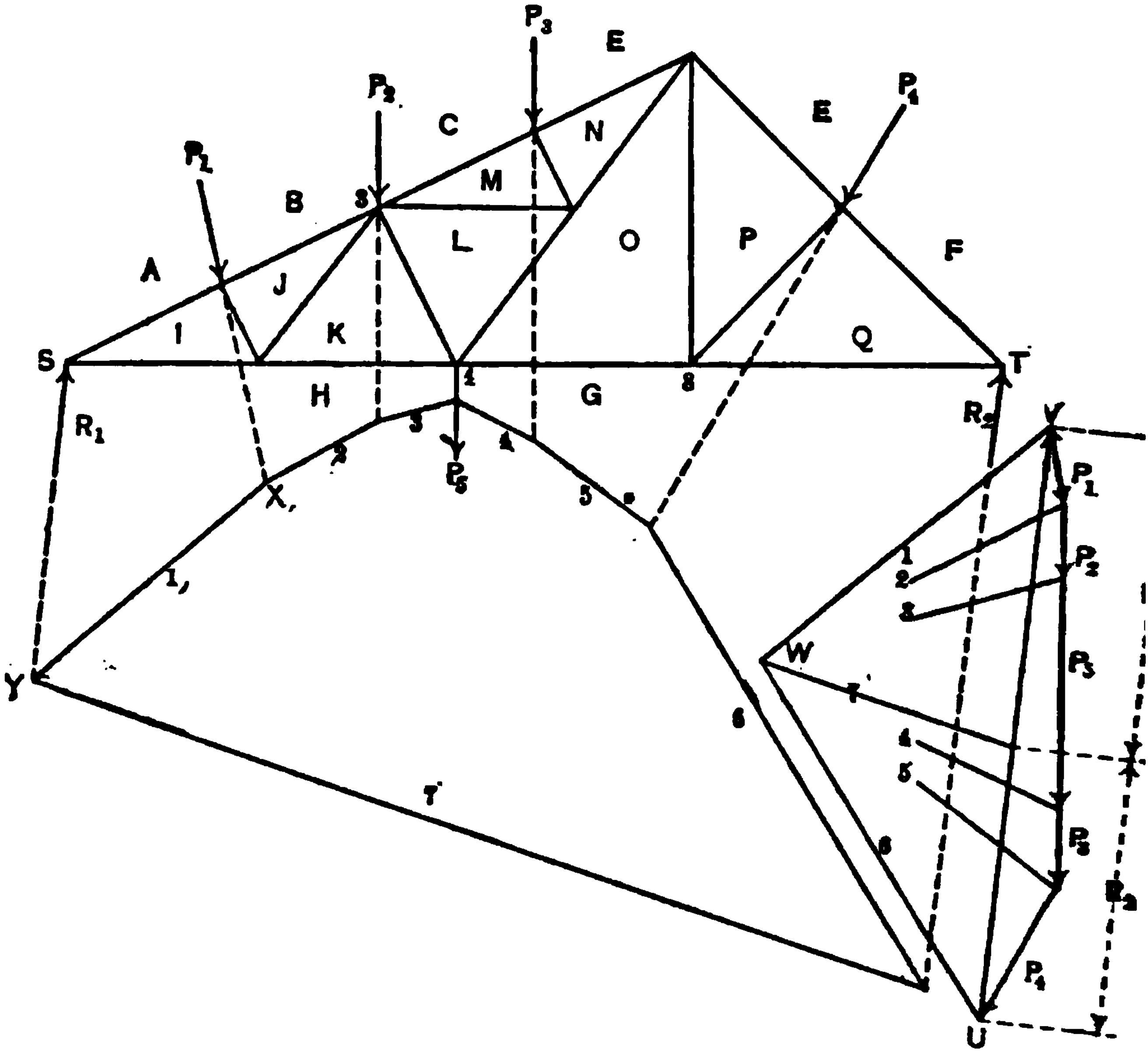


Fig. 38. Unsymmetrical Truss. Truss-diagram

Fig. 38A. Unsymmetri Truss. Force-polygo

forms of trusses SYMMETRICALLY LOADED, it may be well to consider the subj in a more general manner. It will now be assumed that there are no restrictio as to SYMMETRY in the form of the truss and its loading; and, furthermore, will not be assumed that all of the loads act as VERTICAL FORCES as in the proble just solved. Fig. 38 shows an UNSYMMETRICAL TRUSS UNSYMMETRICAL LOADED and with loads or forces which are not parallel. In the previous proble the supporting forces or REACTIONS have been equal and each equal to one-h the load. In this problem such is not the case. The first step, then, is the DETE

THE REACTIONS. If the truss remains in position it follows that
acting upon the truss, such as the LOADS and REACTIONS, must be
UM; also since by definition a TRUSS must act as a BEAM, the truss
aced by a BEAM in considering the OUTSIDE FORCES. In Fig. 38,
lines representing the direction of the forces, as shown, until they
ST and assume ST to be a simple beam loaded with the forces AB,
eginning with AB, to some convenient scale lay off the forces in
vn in Fig. 38A. The broken line VU represents the forces in magni-
rection. For equilibrium, forces equivalent to UV are required.
nt when we remember that the algebraic sum of the vertical and
mponents of all the forces acting must respectively equal zero.
at the supports at S and T, Fig. 38 are similar in every respect we
that the reactions R_1 and R_2 act in the same direction and that they
UV. This does not determine the magnitudes of R_1 and R_2. These
l as follows: In Fig. 38A, assume any point W and draw the lines 1,
Fig. 38, starting at any point on R_1 draw the line 1 parallel to the
38A, and extend it until it cuts the direction or line of action of the
own; from this point draw a line parallel to 2 in Fig. 38A, and extend
the direction of BC as shown, and so continue until line 6 is drawn.

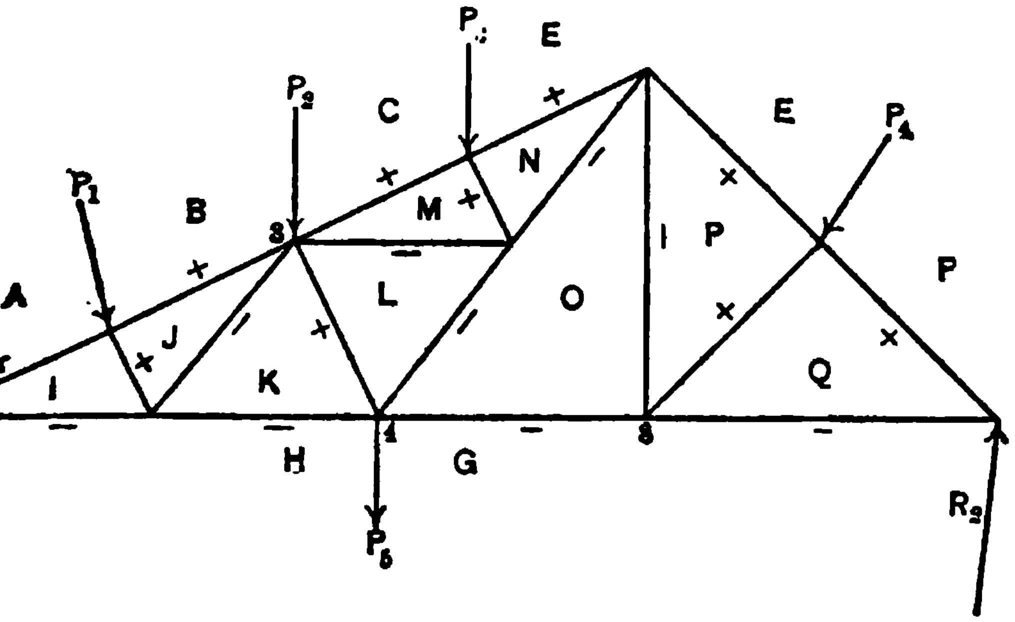


Fig. 38B. Unsymmetrical Truss. Truss-diagram

raw line 7 in Fig. 38 and then in Fig. 38A draw a line parallel to
til it cuts UV. This point divides UV into two parts, the upper
nitude of R_1 and the lower the magnitude of R_2. No trouble will
in applying the above method if the following rule is obeyed to
Fig. 38, the parallels to any three lines in Fig. 38A which form a
meet in a point. For example, in Fig. 38A lines 1, 2, and P_1,
riangle, and in Fig. 38, their parallels meet in the point X. In
is not necessary that the forces AB, BC, etc., be used in order in
onsidering them in order, on a simple beam, makes the graphical
Fig. 38 less complex and avoids many chances of error. The
d above is general and can be used for forces acting in any direc-
rces are parallel then the load-line ab, bc, ce, etc., in Fig. 38A, and
ill coincide; but the method of procedure remains unchanged.
he FORCES acting upon the truss have been determined, for con-
will be shown in character, in Fig. 38B, and the STRESSES in the
osing the truss will be found. First lay off the forces in exact
t they form a CLOSED FIGURE, which must be the case if they are
The lines with arrow-heads in Fig. 38C show this construction,

which checks the values of R_1 and R_2 obtained above. This figure remains t same regardless of the interior arrangement of the truss. The construction the stress-diagram follows the methods given in the previous examples un point 3 is reached. Here there are three UNKNOWNS, *CM*, *ML* and *LK*, and cannot be assumed that *ML* is the same as *JK*, as was done in examples 10 a 11. Let the truss be cut as shown in Fig. 38D, and the actual stresses in the c pieces assumed to act against the cut ends, then the frame shown in Fig. 3 and the forces R_1, *AB*, *BC*, *CE*, *EN*, *NO*, *OG* and *GH* will be in equilibriu

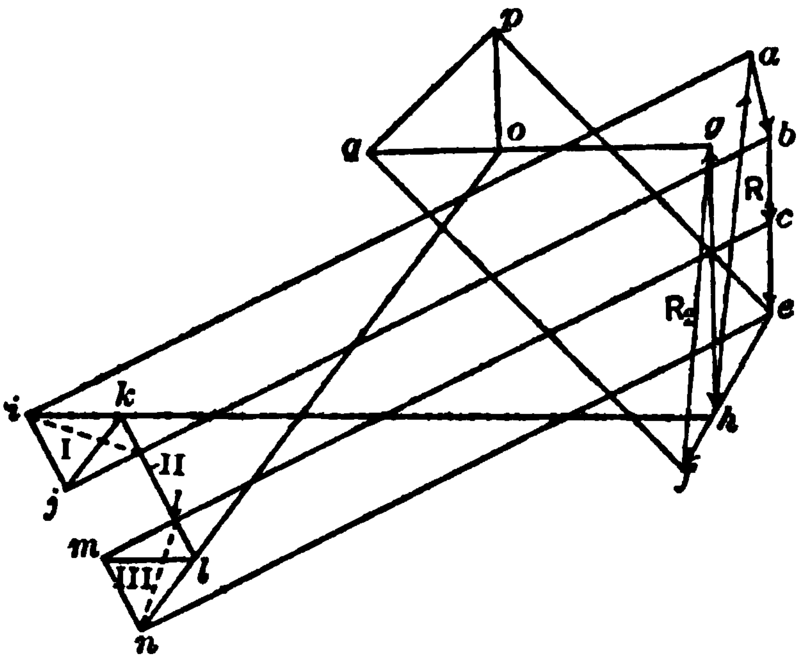


Fig. 38C. Unsymmetrical Truss. Stress-diagram

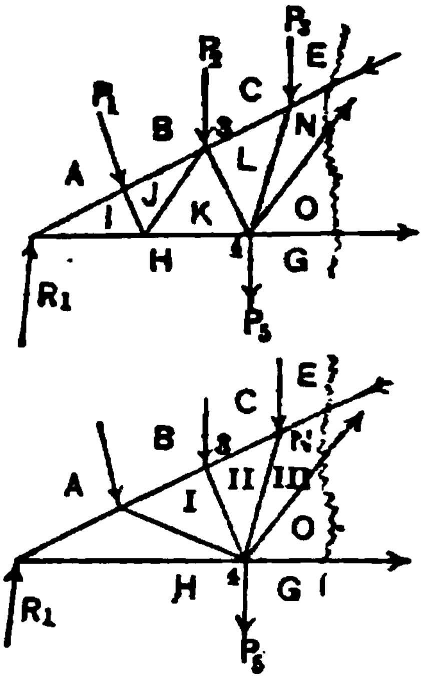


Fig. 38D. Unsymmetrical Tru Truss-diagrams

The frame may be of any form as long as it is rigid so the bracing may be chang as shown in Fig. 38D and the stress-diagram proceeded with as in Fig. 3 until the stresses in *EN*, *NO* and *OG* have been found. This will locate t point *O*. Returning to Fig. 38B, it is found that at joint 4 all the stresses b *KL* and *LO* are known; hence these can be found in the usual manner. Join is next considered and so on until the diagram is complete. The line *qf* in F 38C will pass through *f* if the work is correct. Although the method for det mining the CHARACTER OF THE STRESSES has been explained, it will be repeat here in a more general manner. Take, for example, joint 8, in Fig. 38B, wh is in equilibrium under the action of the stresses *go*, *op*, *pq* and *qg*, as indicated in Fig. 38E. The stress-diagram for this joint is shown in Fig. 38F, separated from Fig. 38C. It is assumed that the stress in *GO* is tension. Then in Fig. 38F, starting at *g* we l off *go*, *op*, *pq* and *qg*, placing the arrow-heads as shown. Transferring th arrow-heads to the ends of the cut pieces in Fig. 38E indicates at once t KIND OF STRESS. The following examples illustrate the above methods.

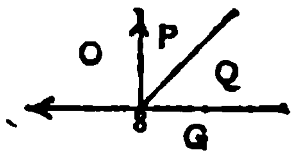


Fig. 38E. Unsymmetrical Truss. Forces at Joint 8

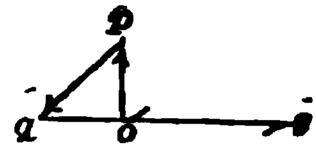
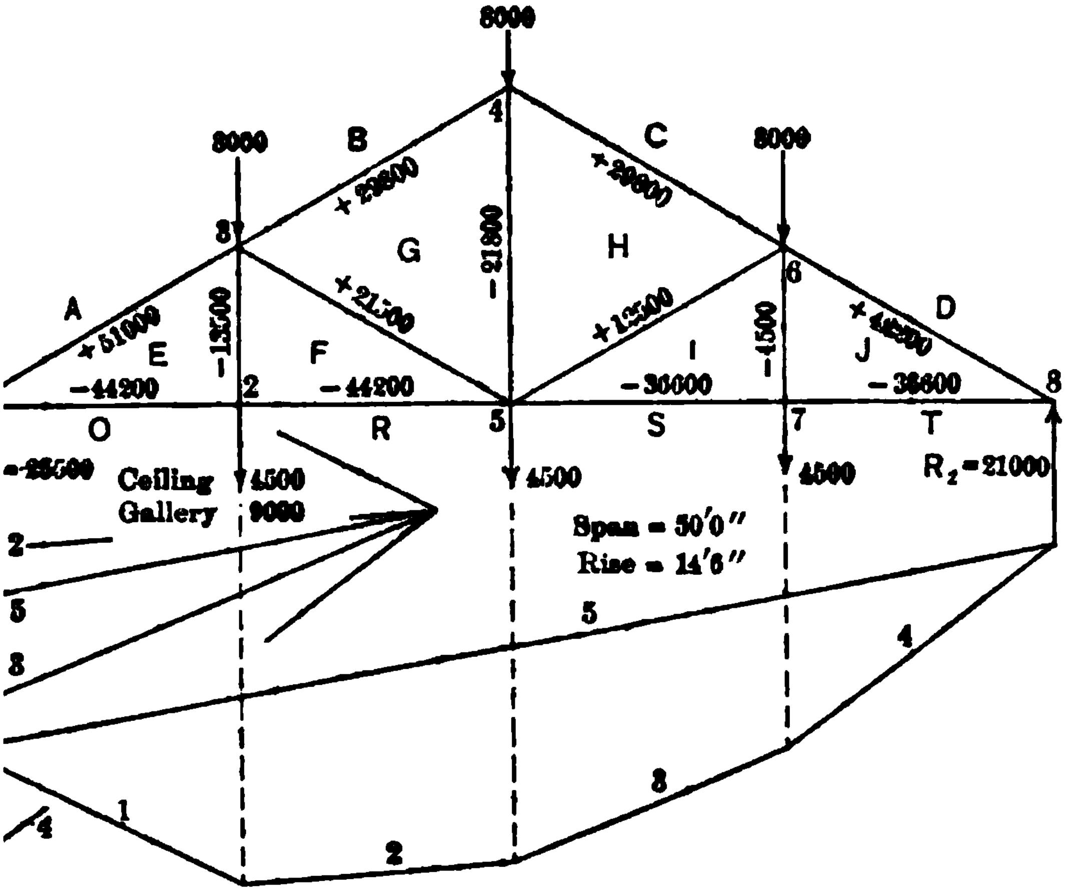


Fig. 38F. Unsymmetri Truss. Stress-polygo

Unsymmetrically-loaded Truss. Example 24. Fig. 39 represents the d gram of a truss similar to that shown in Fig. 1, but of a greater span and hav a gallery supported from it at one side only. The approximate roof and ceili loads are indicated by the figures near the arrows, and the weight coming one truss from the gallery would be about 9 000 lb. The first step tow drawing the stress-diagram is to determine the reactions at the two ends of t

will give the supporting forces. This is readily done in this example
OD OF MOMENTS explained on pages 322 to 324. Moments are first
joint 1. As the loads at joints 2 and 3 have the same arm, they
gether before multiplying by the arm. The loads at joints 4 and 5



39. King-rod Truss. Truss-diagram and Equilibrium-polygon

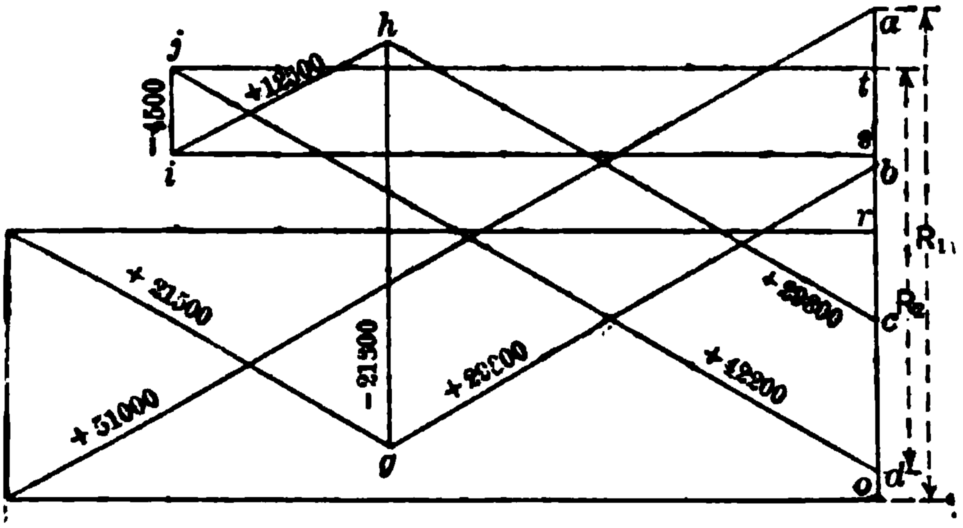


Fig. 39A. King-rod Truss. Stress-diagram

5 and 7 are treated the same way. The moments about joint 1 will

oo + 4 500 + 9 000)	= 21 500] lb × 12½ ft =	268 750 ft-lb
oo + 4 500)	= 12 500] lb × 25 ft =	312 500 ft-lb
oo + 4 500)	= 12 500] lb × 37½ ft =	468 750 ft-lb
The sum of the moments	=	1 050 000 ft-lb

iese CLOCKWISE MOMENTS about joint 1 must be balanced by the
WISE MOMENT of R_2, the LEVER-ARM of which, with reference to
ft. Knowing the arm, 50 ft, the force R_2 is obtained by dividing
e moments of the loads by the span. Dividing 1 050 000 ft-lb by

50 ft, the result is 21 000 lb, which is the reaction or supporting force at joint and R_1 must equal the difference between the sum of the loads and R_2. Th sum of the loads is 46 500 lb and subtracting from this 21 000 lb, the remainde 25 500 lb, is the value of R_1. The stress-diagram (Fig. 39A) may now be draw First draw a vertical line *oa* equal to R_1, 25 500 lb. From *a* and *o* draw lin parallel respectively to *AE* and *EO*, locating the point *e*. For the stress-lin at joint 2 measure up from *o* a distance equal to the load at that joint, 13 500 l which gives the point *r*, and from *e* and *r* draw lines parallel to *EF* and *F* which intersect at *f*. At joint 3, the sides of the stress-polygon are *fe*, *ea*, *bg* and *gf*. Draw the stress-polygons for joints 4, 5, and 6 in the order in whic

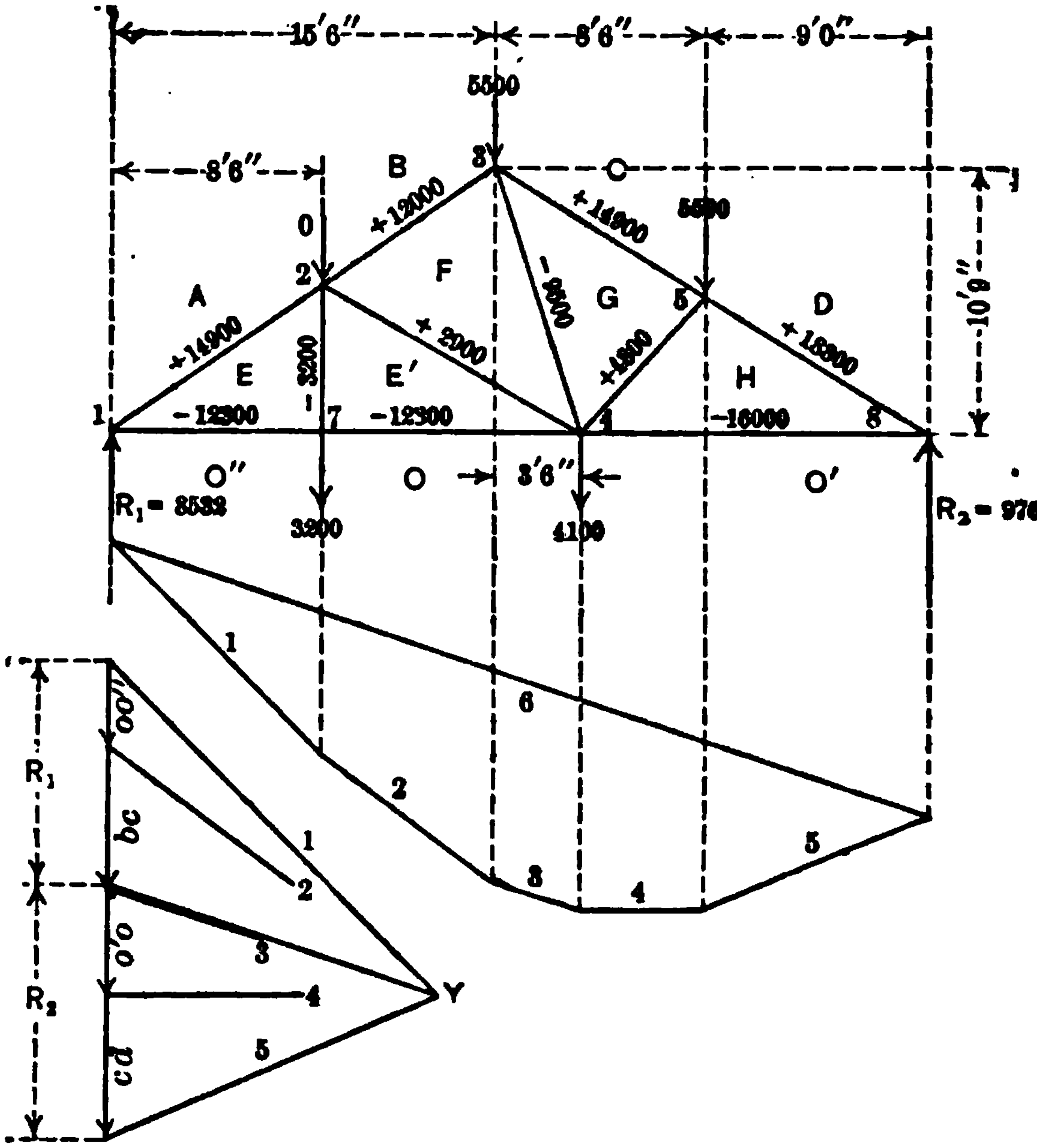


Fig. 40. Unsymmetrical Truss. Truss-diagram and Equilibrium-polygon

they are numbered. At joint 6 the sides of the stress-polygon are *ik*, *kc*, *cd*, and *ji*. If the diagram has been correctly drawn, the line *ij* will be just equ to the load at joint 7. The sides of the stress-polygon for joint 7 are *ts* equal t 4 500 lb, *si*, *ij* and *jt*, the only line to be drawn being *jt*, which must be paral to *JT*. Consequently *j* must be exactly opposite *t*, or the polygon will not clos The distance *dt* should be equal to R_2.

Unsymmetrically-loaded Truss. Example 25. Fig. 40 is the diagram c a wooden roof-truss. The actual loads were about as given on the diagram There were purlins at joints 3 and 5 only, and the ceiling below was suspende by rods from joints 4 and 7, joint 4 being fixed by the framing of the ceilin The moments of the loads about joint 1 are:

3 200 lb × 8½ ft = 27 200 ft-lb
5 500 lb × 15½ ft = 85 250 ft-lb
4 100 lb × 19 ft = 77 900 ft-lb
5 500 lb × 24 ft = 132 000 ft-lb

Sum of moments = 322 350 ft-lb

: sum of the moments by the distance between the supporting forces, s 9 768 lb as the value of R_2. : the loads is 18 500 lb. Sub- 768 lb, 8 532 remains as the . To draw the stress-diagram, $o''a$ equal to 8 532 lb equal raw ae and eo''. The sides of olygon for joint 2 are ea, ab, bf, At joint 3, fb is known and red down and made equal to ; and gf are then drawn. At t by measuring upwards from locating point o' and draw gh t joint 5, hg and gc are now uals 5 500 lb and a line from d allel to DH. This should pass through h, completing the diagram. rod 2–7 is the load at joint 7.

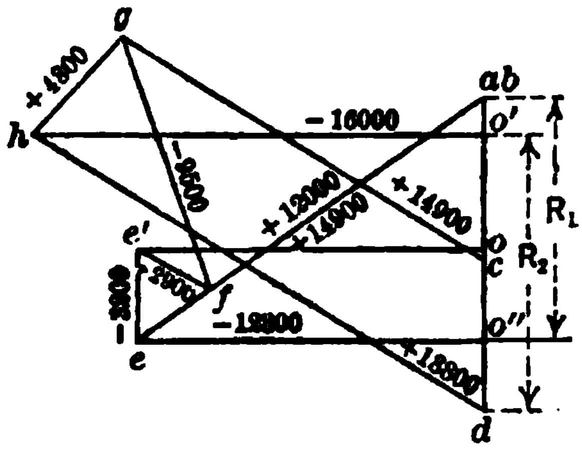
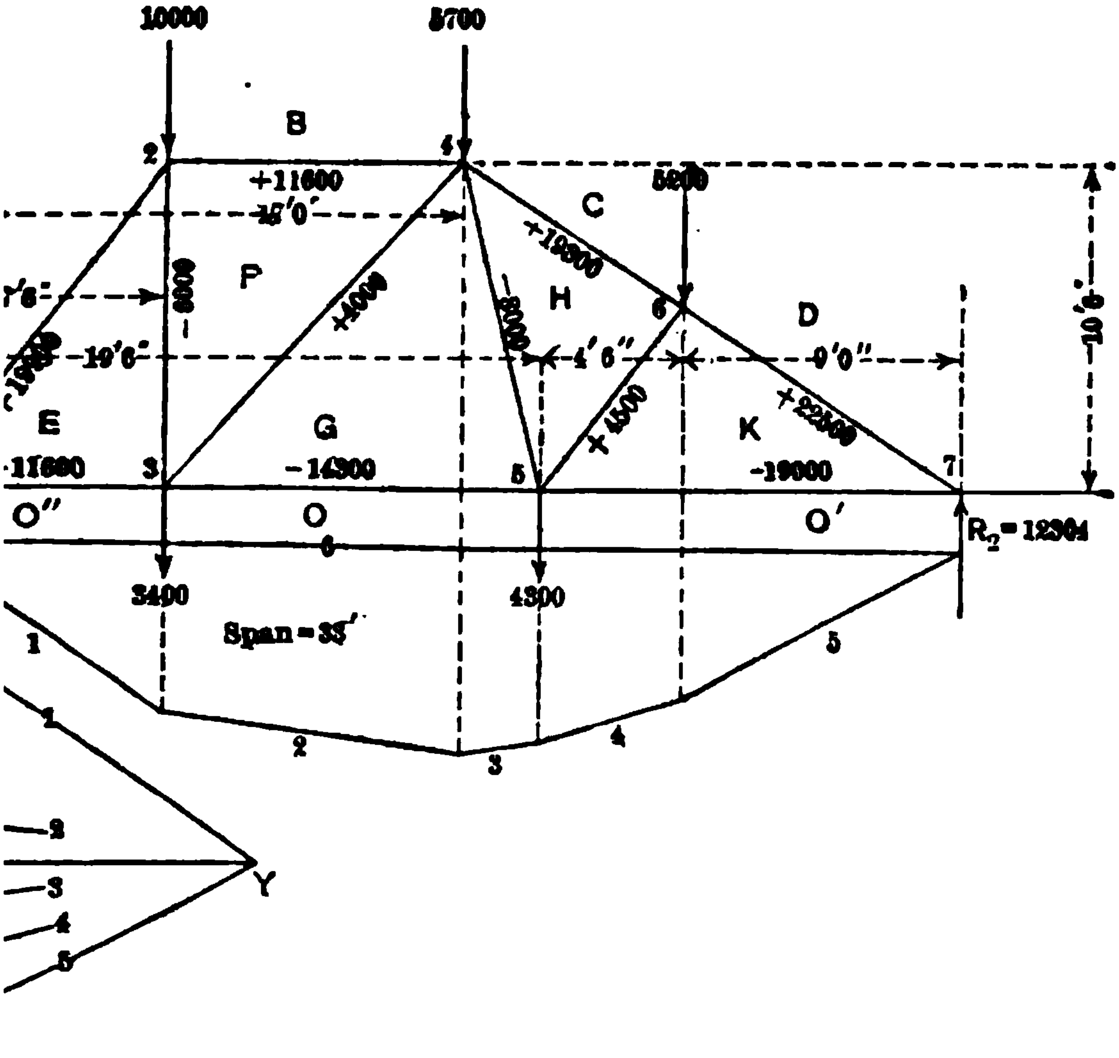


Fig. 40A. Unsymmetrical Truss. Stress-diagram



. Unsymmetrical Truss. Truss-diagram and Equilibrium-polygon

rically-loaded Truss. **Example 26.** Fig. 41 is the truss-diagram ss in the same building in which the truss shown in Fig. 40 was moments about joint 1, there results, for the sum of the moments,

[illegible] and dividing this by [illegible] it gives [illegible] as the value of R_2. T[illegible] sum of the loads [illegible] which [illegible] is the value of R_1. T[illegible] stress-diagram Fig. 41A is drawn in the same manner as in Fig. 40. Starti[illegible] with [illegible] equal to R_1, [illegible] is drawn equal to the load at joint [illegible], and the act[illegible] stress in [illegible] is found by [illegible] the length of the line [illegible]. If the stress-diagram correctly drawn a line through [illegible] parallel to KD will pass through the point previously determined. The CHARACTER OF THE STRESSES is indicated by t[illegible]

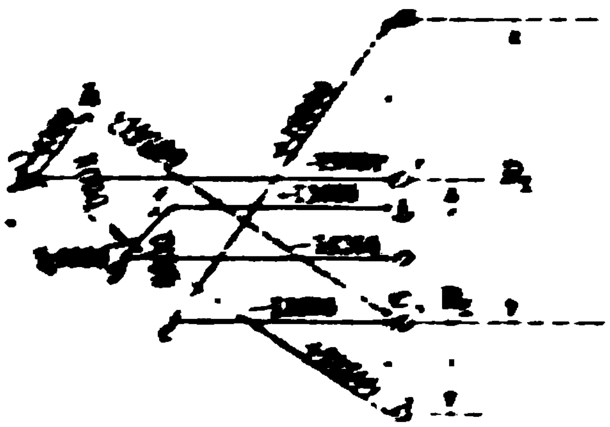


Fig. 41A. Unsymmetrical Truss. Stress-diagram

PLUS AND MINUS SIGNS in Fig. 41, p[illegible] denoting compression. If the stress-d[illegible] grams in the last three examples a[illegible] compared with those for symmetrica[illegible] loaded trusses of similar shape it is fou[illegible] that while the stress-diagrams, Figs. 3[illegible] 40A and 41A are unsymmetrical, th[illegible] are of the same general character, a[illegible] the stresses are all of the same kind when the supporting forces are equ[illegible] This condition holds true for most tri[illegible] gular trusses, but for trusses with ho[illegible] zontal or curved chords, unsymmetri[illegible] loading usually causes a REVERSAL OF THE STRESS IN KIND in one or more of t[illegible] diagonals or verticals; and if the truss contains any four-sided panels, an ad[illegible] tional diagonal is generally required. This is particularly true of the Ho[illegible] TRUSS; and as this truss is very extensively used by architects and builders, will now be considered at some length with special reference to the effect UNSYMMETRICAL LOADING.

Howe Trusses, Unsymmetrically Loaded. When a HOWE TRUSS is load[illegible] symmetrically on each side of the middle, all of the braces incline downwa[illegible] from the center, as in Figs. 14 to 17, Chapter XXVI; and if there is an e[illegible] NUMBER OF PANELS, the middle panel needs no brace. When a load of mu[illegible] magnitude is placed on one side of a truss having an ODD NUMBER OF PANE[illegible] without a corresponding load on the other side, a brace is always required the middle panel and the brace should incline downward from the side which is most heavily loaded.

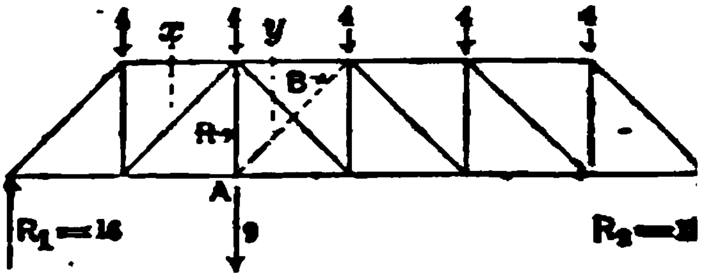


Fig. 42. Howe Truss. Truss-diagram

Howe Truss with Even Number of Panels. When the truss has an EVEN NUMBER OF PANELS, an unsymmetrical load causes a greater stress in the braces on one side of the truss than on the other; and if there is a sufficient difference in the loads on the two sides of the truss, it causes compressive stres[illegible] in one or more of the rods and tensile stresses in one or more of the braces. [illegible] this truss is especially designed with the idea of having the BRACES IN CO[illegible] PRESSION and the VERTICALS IN TENSION, whenever the loading causes tensi[illegible] in a brace, or compression in a rod, the direction of the brace should be revers[illegible] causing it to be in compression again. Consider, for example, the truss sho[illegible] in Fig. 42, divided into 6 panels of equal width and loaded with 4 tons at ea[illegible] of the upper joints and 9 tons at the second lower joint from the left. Wi[illegible] out the bottom load of 9 tons, the brace in the third panel should incline do[illegible] ward from the middle joint, as shown by dotted line at B; but when the load 9 tons is added, it causes a tensile stress in B and a compressive stress in R.

he DIRECTION OF THE BRACE IS REVERSED, as shown by the full line: compression and the vertical R has no stress except that caused by ad of 9 tons. There are the same results when the load of 9 tons the joint directly above, instead of at the lower joint, although in e is no stress at all in R except that due to the weight of the tie-beam. ad of 9 tons is reduced to 6 tons, no brace is required in the third when the bottom load i tons, a brace in the tion is required, as 43. (See page 1006.)

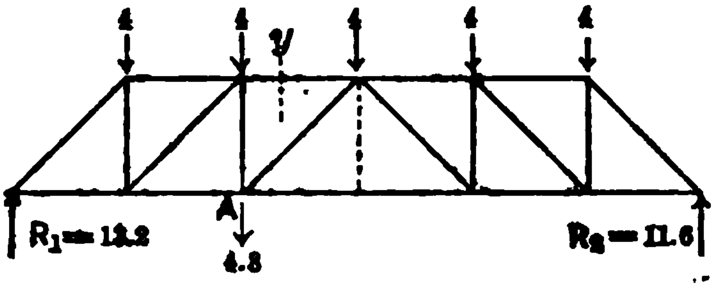
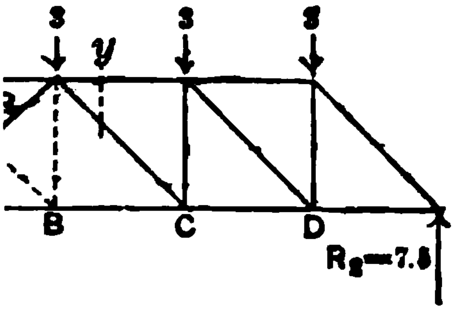


Fig. 43. Howe Truss. Truss-diagram

russ with Uneven Panels. In the five-shown in Fig. 44, a tons at A requires nent of braces shown lines, and when the load at A is increased to more than 15 tons, needs to be reversed, as shown by the dotted line. The stress-ys shows in which direction any brace should be placed to be in but this may be determined also by the following rule. When the ads to the left of any section, taken between R_1 and the middle, is the reaction R_1, the direction of the brace cut by that section must rom its normal direction. When the sum of the loads is less than should be in its normal position. When the sum of the loads, to e section, is just equal to R_1, no brace is required. For example, consider a section at x, Fig. 44. Here the sum of the loads to the left is 10.5 tons, which sum is less than R_1; hence the brace should be in its normal direction. If the section is at y, the sum of the loads to the left is 13.5 tons, which sum is greater than R_1; hence a brace is required, slanting downward from the more heavily loaded side. When the sec-



we Truss. Truss-diagram

Fig. 42, the load to the left is 4 tons, an amount less than R_1; ce in that panel should be in its normal position. When the sec-he sum of the loads is greater than R_1; hence the brace in that reversed. When the section is at y, Fig. 43, the sum of the ft is less than R_1; hence the brace should be in its normal posi-s rule the proper direction of the brace in any panel is indicated, he complication of the loading and of the width of the panels; but he rule, it is first necessary to determine the supporting forces, found either by the METHOD OF MOMENTS, as explained in r by the GRAPHICAL METHOD.

rical Howe Truss. Example 27. As an example of an unsym-E TRUSS unsymmetrically loaded, the truss represented in Fig. 45 This truss is supposed to support a flat roof and a wooden tower rn. The position of the tower necessitates a division of the panels o that the truss is quite unsymmetrical. It is assumed that the oof, snow, and tower constitute the loads in pounds at the upper d by the figures. The graphical determination of the reactions clearly shown in Figs. 45 and 45A. The only panels of this truss is any question as to the direction of the braces are the third and

fourth. Taking a section at x, the sum of the loads to the left is greater th R_1; hence the brace should be placed as drawn. A section taken through makes the sum of the loads to the left less than R_1; and hence the brace shou be in its normal position. The stress-diagram of this truss is readily draw starting with wa equal to R_1, and going from joint to joint as in previous exa ples. The completed stress-diagram is shown in Fig. 45A.

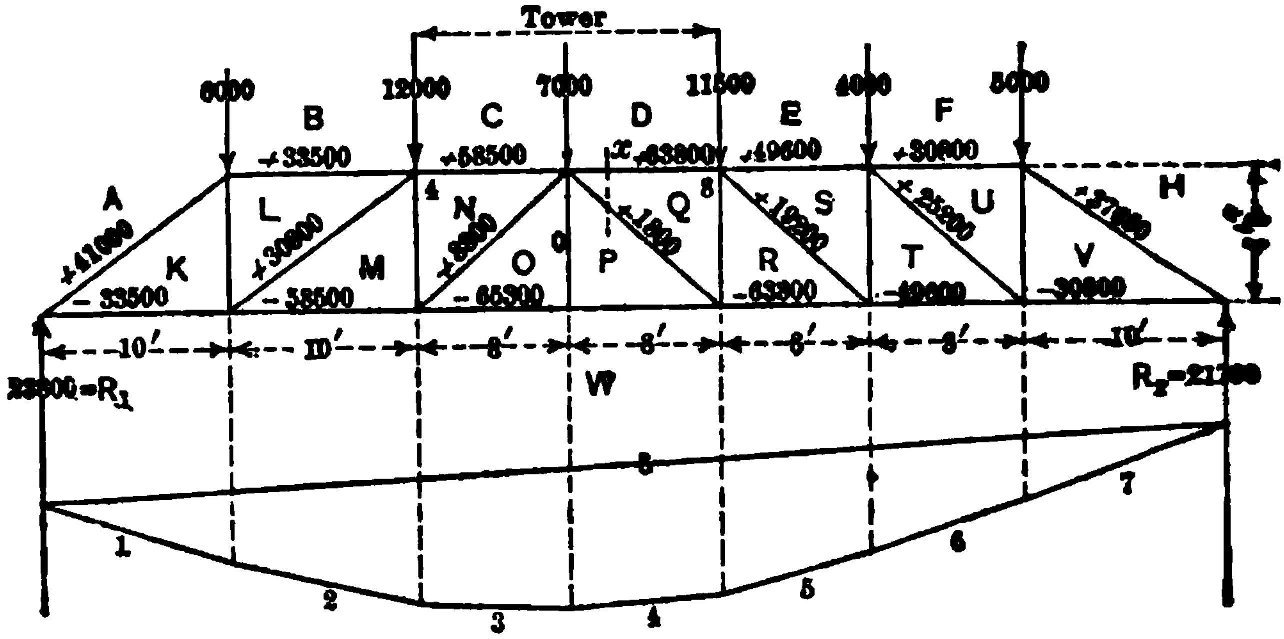


Fig. 45. Howe Truss. Truss-diagram

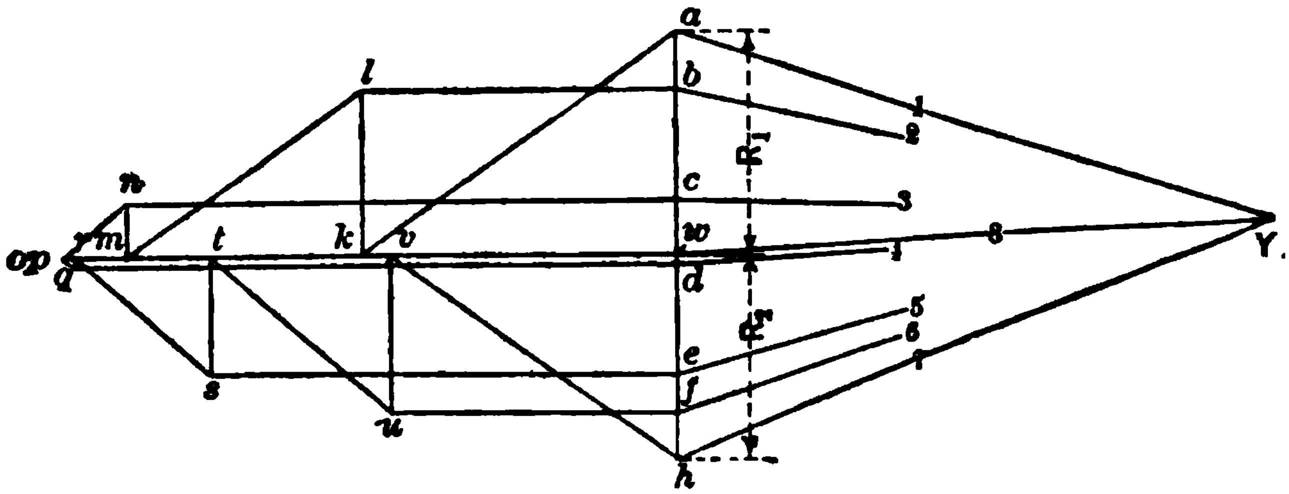


Fig. 45A. Howe Truss. Stress-diagram

Counterbraces. These are EXTRA BRACES that are put in a truss whe stresses are REVERSED IN CHARACTER by a load which may be applied for a tim and then removed. For illustration, consider the truss represented in Figs. 4 and 43. Here it has already been shown that when the load at A is less than 6 the brace in the third panel should be in the position shown in Fig. 43; whil when the load is greater than 6, the brace should be in the position shown b the full line, Fig. 42. Now, if the load at A represents the weight of a crowde gallery or a hoist raising a heavy load, or in fact if it represents any live load it is evident that when this live load is absent the brace in the third panel shoul be in its normal position; and that when this maximum load is present a brac is needed in the opposite direction. As it is not practicable to move the brac to suit the changing conditions of the loading, it is necessary to put in tw braces, only one of which, however, is in action at a time. The stress in a HOWE TRUSS, therefore, which is subject to a variable and unsymmetrical loading, should be computed for at least TWO CONDITIONS OF LOADING: first for the condition resulting from the APPLICATION OF THE MAXIMUM LOAD: an

or the condition resulting from the REMOVAL OF THE VARIABLE LOAD. should be designed to resist both conditions. SNOW is a VARIABLE hich such trusses are often subjected; but as it is nearly uniformly over the roof, it does not change the CHARACTER OF THE STRESSES he members. If a truss, therefore, is designed for a MAXIMUM SNOW- more than strong enough when there is no snow. Moreover, the strength of the chords is usually sufficient to resist any slight inequality ling. The principal VARIABLE VERTICAL LOADS, therefore, to which s may be subjected and which require counterbraces, are those due ht of people, merchandise, etc., these loads being either suspended uss by rods or brought upon the truss by a floor supported by the rd. The truss shown in Fig. 45, also, is an instance of such loading. s given by the figures indicate merely the combined dead loads and During a high WIND the weight on the LEEWARD SIDE of the tower reased and on the WINDWARD SIDE decreased, so that when the wind the right, the load at 4 is greater and at 8 less than indicated; while ind blows from the left the load is increased at 8 and decreased at 4. es COUNTERBRACES in both the third and fourth panels. As counter- o harm, even if never brought into action, it is always well to use middle panels wherever the loads are at all variable.

er Trusses. These trusses may be considered as UNSYMMETRICALLY SSES, for although the loads may be symmetrical in relation to the ure usually unsymmetrical in relation to the supports. The method G THE SUPPORTING FORCES and drawing the stress-diagrams is shown wing examples:

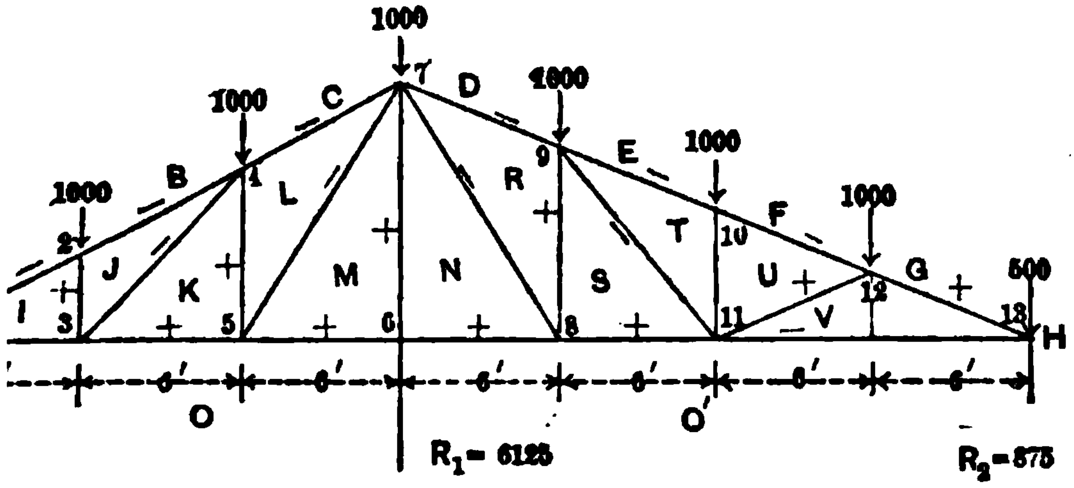


Fig. 46. Cantilever Truss. Truss-diagram

r Truss. Example 28. Fig. 46 is the diagram of a CANTILEVER s might be used to support the roof over a grand stand or railway- orm and may be constructed either of wood or steel, steel being The first step towards determining the stresses is to find the sup- s. For this purpose the panel-loads have been made 1 000 lb each, ng of equal width. These assumed loads simplify the problem and as the actual loads to explain the method of procedure. In CANTI- ES the loads at the ends of the trusses, as well as the intermediate be taken into account. These end-loads are each equal to one-half loads. To find the supporting forces moments are taken about e sum of the moments of the external vertical forces is 147 000 ft-lb. its must be resisted by the moment of the force R_1, which acts in rection with reference to the same point and with a lever-arm of ing 147 000 ft-lb by 24 ft, there results 6 125 lb as the value of

and as the total load is 7 000 lb, R_2 must be 875 lb. The stress-diagram may be commenced either with the forces at joint 1 or with those at joint 13; but as the external loads were laid off from left to right in the preceding examples, the same order is used here. Commencing then with joint 1, lay off on a vertical line the load *oa* equal to 500 lb, which acts down, and draw *ai* and *io* parallel respectively to *AI* and *IO*. The forces act from *o* to *a*, from *a* to *i* (from the joint) and from *i* to *o* (toward the joint), showing that *AI* is in tension and *IO* in compression, a REVERSAL OF THE CHARACTER OF THE STRESSES developed in the corresponding members of a truss supported at both ends. Next, at joint 2, the stress *ia* is now known, and *ab* equal to 1 000 lb is laid off; then *bj* and *ji* are drawn, *BJ* being in tension and *JI* in compression. The forces at joint 3 are next drawn and then those for the remaining joints, in the order in which they are numbered. At joint 6, the first force known is the supporting force R_1, represented by *o'o* laid off equal to 6 125 lb and acting upward. The sides of the polygon of forces for joint 6 are *o'o*, *om*, *mn* and *no'*. The stress in *MN* is equal to the supporting force *o'o*, which is evident from the truss-diagram. In practice, R_1 would probably be a COLUMN continued to the apex of the truss. At joint 12 the stresses already determined are *vu*, *uf*, and *fg* equal to 1 000 lb. *gv* must close the polygon. It will be noticed that *gv* acts toward joint 12; hence the rafter in the end-panel is in compression. If a line drawn from *g* parallel to the rafter, passes through *v*, the stress-diagram is correct; if it does not pass through *v*, then either the stress-diagram has not been drawn with

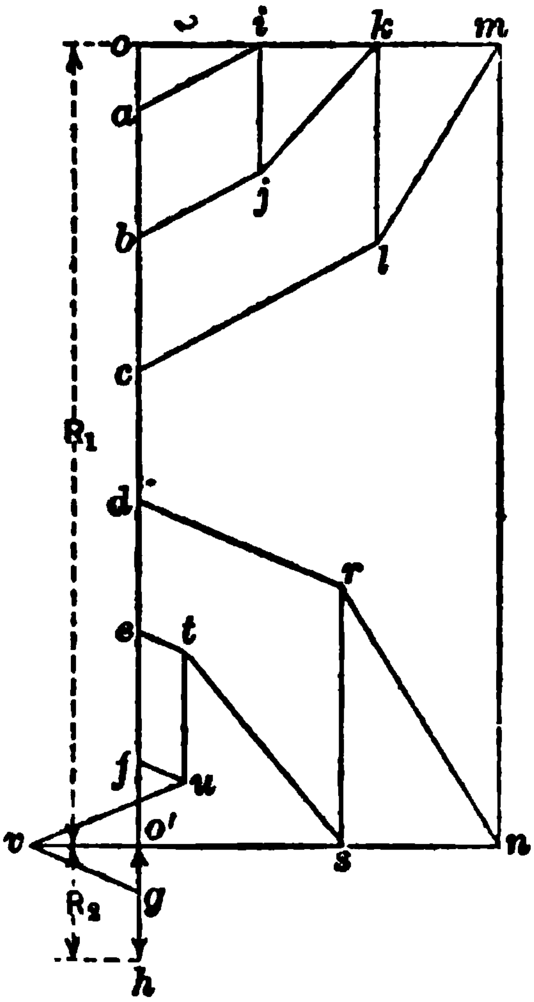


Fig. 46A. Cantilever Truss. Stress-diagram

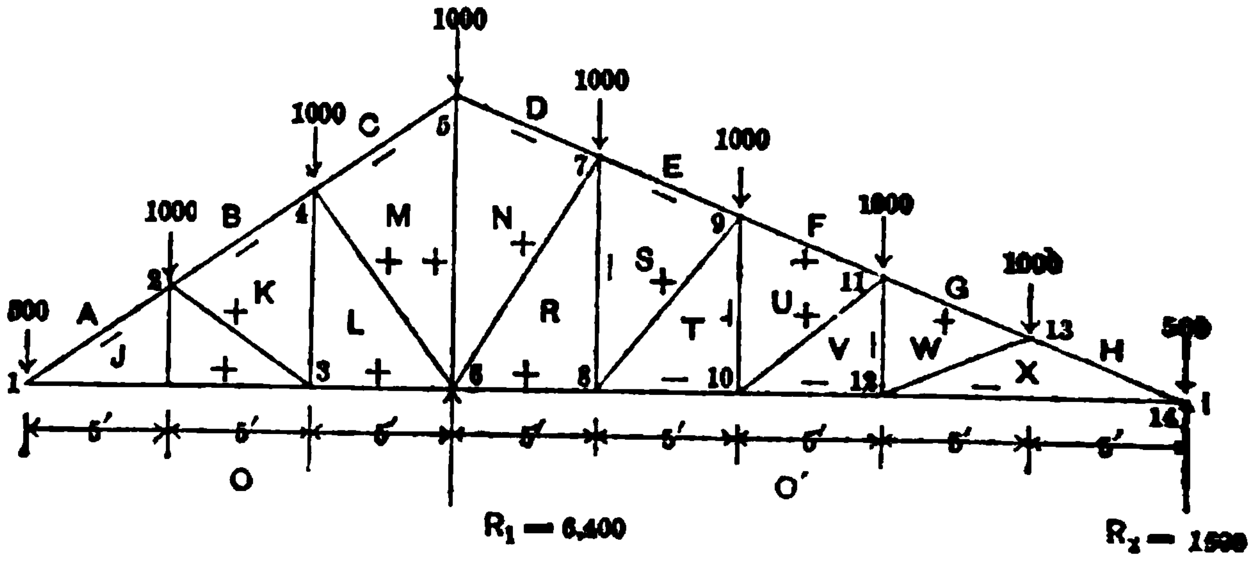


Fig. 47. Cantilever Truss. Truss-diagram

sufficient accuracy or an error has been made in computing the supporting forces. In drawing the stress-diagram for CANTILEVER TRUSSES, it is important to keep in mind THE DIRECTION IN WHICH THE FORCES ACT, in order to determine which members are in compression and which in tension.

r Truss. Example 29. Fig. 47 is the diagram of a truss similar that shown in Fig. 46, but with the DIAGONAL BRACES INCLINED SITE DIRECTION, so as to cause them to be in compression and the tension. The supporting forces are found by the same methods mple 28, and the m, also, is drawn by used for Fig. 46A. however, the stress l post MN is con- than the reaction a large portion of ransmitted to joint uts LM and NR. three sections of the right side are on and three sec- bottom chord are This is because in projection of the proportion to the less than it is in n the stress-lines t of the load-line he STRESSES ARE KIND. This truss pted to wooden vith vertical rods truss shown in

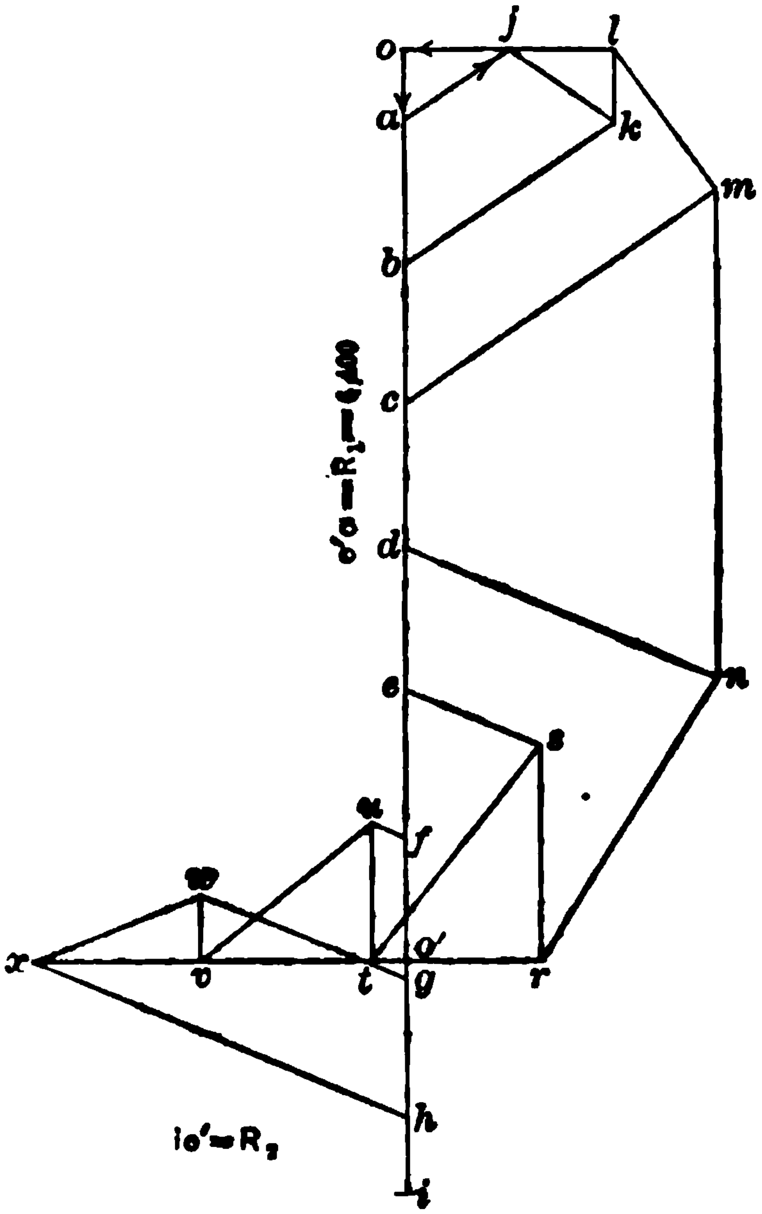


Fig. 47A. Cantilever Truss. Stress-diagram

antilever Truss. In this example, wn a truss with at the outer end , so that R_1 acts determine the character of the supporting forces moments are taken about rs:

nts of loads to the right of joint 7, the figures on the force-arrows gram indicating thousands of pounds:

$$8) + (5 \times 16) + (5 \times 24) + (12.5 \times 32) = 640\,000 \text{ ft-lb}$$

nts of loads to the left of joint 7:

$$(2.5 \times 24) + (5 \times 16) + (5 \times 8) = 180\,000 \text{ ft-lb}$$

its act in opposite direction with reference to the center o. 6, the smaller sum, is subtracted from the larger, leaving an ment of 640 000 ft-lb − 180 000 ft-lb = 460 000 ft-lb, tending down on the right of R_2 at joint 6 and to lift it up on the left. st be resisted by the moment of the reaction R_1, which has an viding 460 000 ft-lb by 24 ft, 19 250 lb results as the reaction equires a downward force of this magnitude to maintain the um. As the support at 6 must resist this downward pull as R_2 will equal the sum of the loads plus the pull R_1, or 45 000 lb

+ 19 250 lb = 64 250 lb. Having obtained the value of the supporting forc the stress-diagram is drawn by laying off on a vertical line *oa* downward, equ

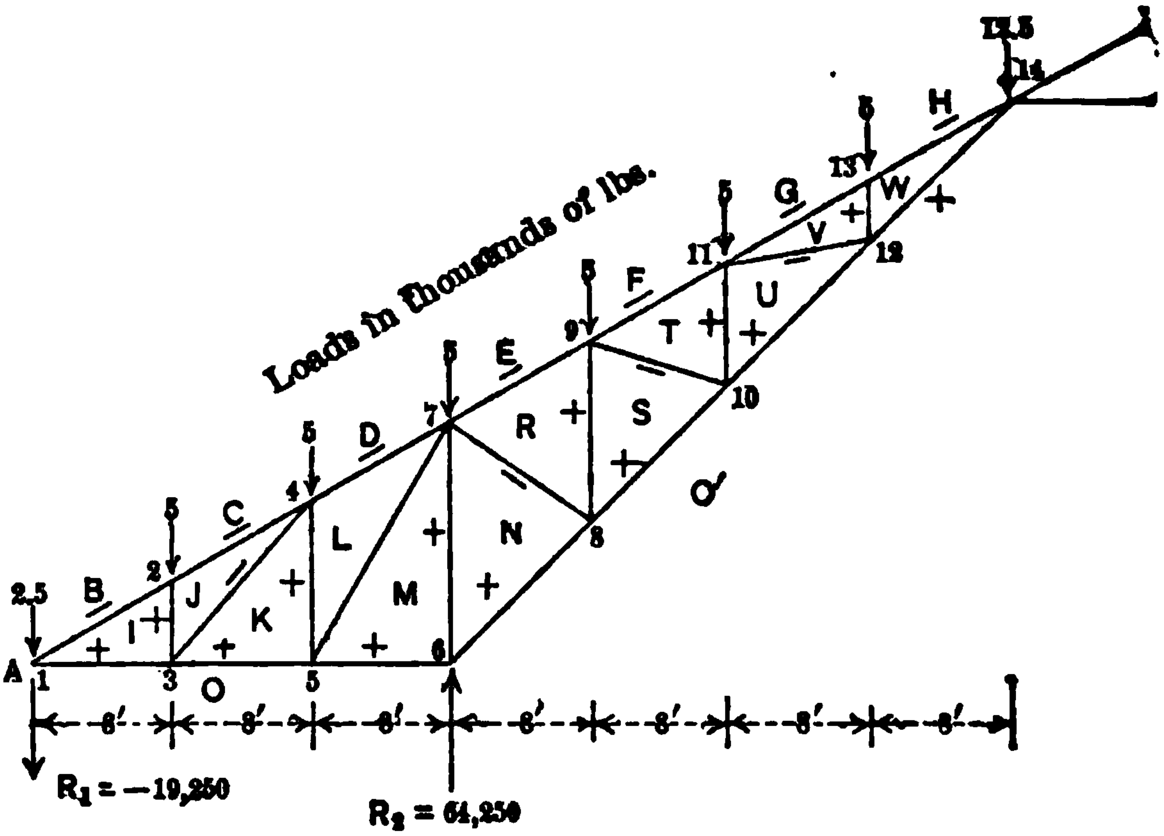


Fig. 48. Cantilever Truss. Truss-diagram

to 19 250 lb equal to R_1. The next force is the load of 2 500 lb, which also a down, and which locates the point *b*. From *b* a line parallel to *BI* is drawn a from *o* a line parallel to *I* locating the point *i*. *bi* a from joint 1 and *io* towa it, showing that *BI* is tension and *IO* in compressi The remainder of the stre diagram is drawn by the sa methods employed for diagrams of Figs. 46A a 47A. At joint 6 the for polygon is begun with force R_2 or *o′o*, which a upward, and the upper e of which must be at *o*. Co sequently *o′* is located measuring downward from 64 250 lb. The sides of stress-polygon for this jo are *o′o*, *om*, *mn* and After *gh*, the load at joint is laid off, the remaining d tance *ho′* should be just eq to the load at joint 14, 12 500 lb. If R_1 and R_2 ha been correctly computed and the stress-diagram accurately drawn, the poi *s*, *u* and *w* will fall in the line *no′*.

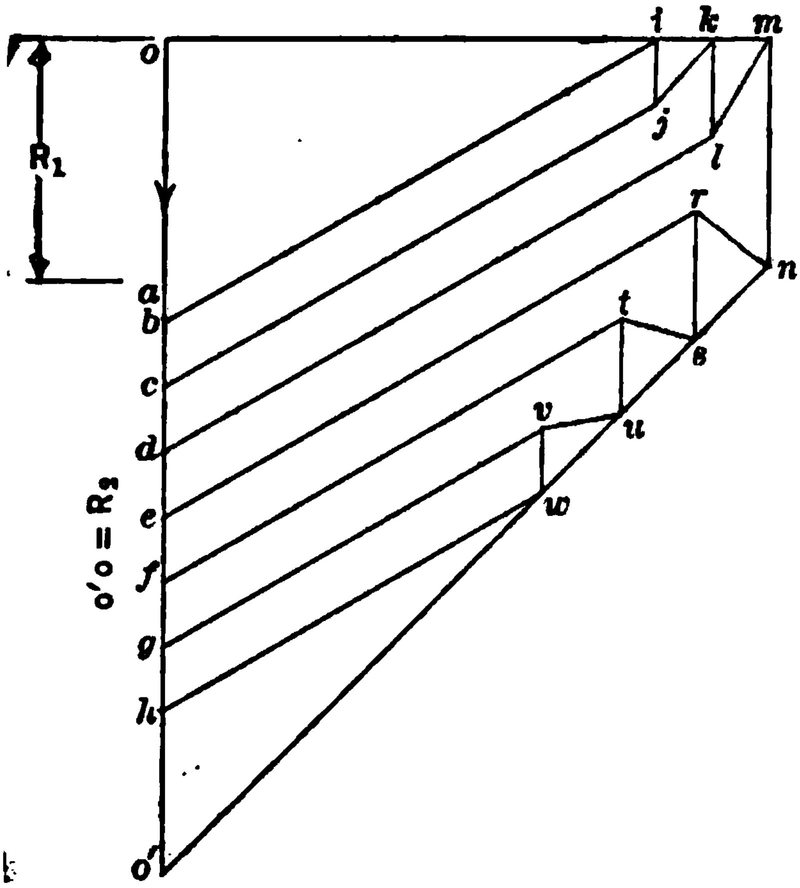


Fig. 48A. Cantilever Truss. Stress-diagram

6. Determination of Wind-Load Stresses

ads. Thus far the stresses due to VERTICAL LOADS only have been :he pressure of the WIND being combined with the DEAD LOAD and ıs acting vertically. For TRIANGULAR and FINK TRUSSES this ıfficiently accurate, as the wind-pressure never causes a maximum ss of that obtained by the method explained in connection with the ımples. For TRUSSES WITH CURVED CHORDS and in fact for almost STEEL TRUSSES except those of the FINK and FAN TYPES, it is not der wind-pressure as acting vertically, because the wind acts gen- rection at right-angles to the roof-surface, and upon but one side of given time, thus loading the truss unsymmetrically and often caus- of an opposite kind from those produced by a vertical loading. ı are inactive under a vertical load may therefore be necessary to ce of the wind, or the total stress due to wind and vertical load y be greater than it would be if the wind-pressure were considered load. To design a roof-truss correctly, therefore, it is necessary the stresses due to VERTICAL LOADS and WIND-LOADS separately bine them so as to get the GREATEST STRESS that may be produced obable conditions. (See statement on page 1049.)

ıords. In the calculation of trusses with CURVED CHORDS it is ctice to find the stresses for the following different loadings and them to obtain the maximum stress: Stresses due to the wind on e truss nearer the expansion-end; stresses due to the wind on the ss nearer the fixed end; stresses due to the permanent dead loads; snow covering the entire roof or only one-half of the roof; and, in stresses due to snow covering only a small area of the roof on one

Snow. It is generally assumed that the maximum wind-pressure load can not act on the same half of the truss at the same time. h straight rafters it will generally be sufficient to find the stresses nanent dead load, and to the wind from both directions, disregard- ad when the pitch of the roof is 45° or greater. For the Northern ıe pitch is less than 30°, it is well to consider that a heavy sleet h sides of the roof at the time of a heavy wind and to add about f roof-surface to the dead load to allow for it. In localities where s may be expected, the stresses due to the full snow-load should as these combined with the permanent dead load may exceed ad load, sleet and wind-pressure.

-Diagrams. These are affected by the manner in which the truss If both ends of the truss are fixed, the WIND-REACTIONS are paral- ant wind-load; if one end is free to move horizontally, that is, supported on a ROCKER, the reaction at the roller-end is vertical fixed end inclined. "If one end be fixed and the other merely a smooth IRON PLATE, the reaction at the free end may have a onent equal to the vertical component multiplied by the COEFFI- ON, which is about one-third."

'ree Ends of Trusses. Wooden trusses may be considered as ds. STEEL TRUSSES, when supported on masonry walls, should XED and the other FREE to move; and when the span exceeds d should be supported on ROLLERS to permit of expansion or hen steel trusses are supported by steel columns, as in steel mill- usses are RIGIDLY ATTACHED to the columns and no provision

is made for expansion. In such buildings the wind-pressure causes a BENDI STRESS in the columns, which must be provided for.

Truss with Fixed Ends. Example 31. Wind-pressure is usually assum to be applied uniformly over one side of the roof and to act at right-angle the surface of the roof. The joint-loads or panel-loads, therefore, are proportio to the roof-areas supported. When the joints divide the rafter into panels of eq length, the joint-loads are uniform, except for the joints at the edges of the r The actual wind-pressure is obtained by multiplying the roof-surface by values given in Table IX, page 1053. For this example the triangular truss sho in outline by Fig. 49 is considered and it is assumed that the span and spac of the truss are such as will give a load of 1 000 lb at joints 2 and 4. The lo at joints 1 and 5 are only one-half of those at 2 or 4. To find the support forces or reactions, draw a line representing the resultant of the loads, cutt the bottom chord at X. As the loads are symmetrical the resultant acts at middle of the rafter and at right-angles to it. The reactions R_1 and R_2 inversely proportional to the two segments into which a horizontal line join the points of support is divided by the resultant, or in this case to X–7 and 1-

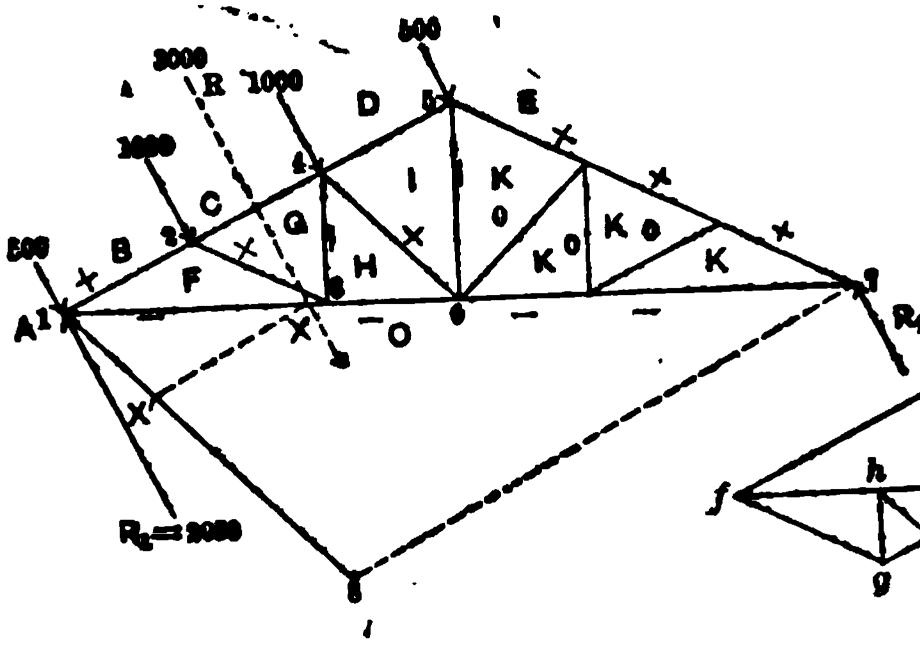


Fig. 49. Triangular Truss. Truss-diagram

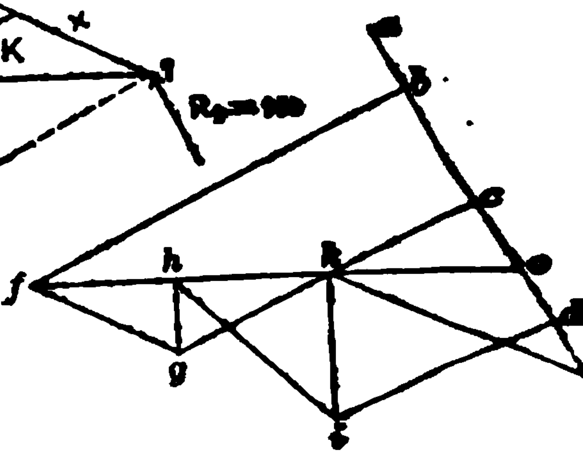
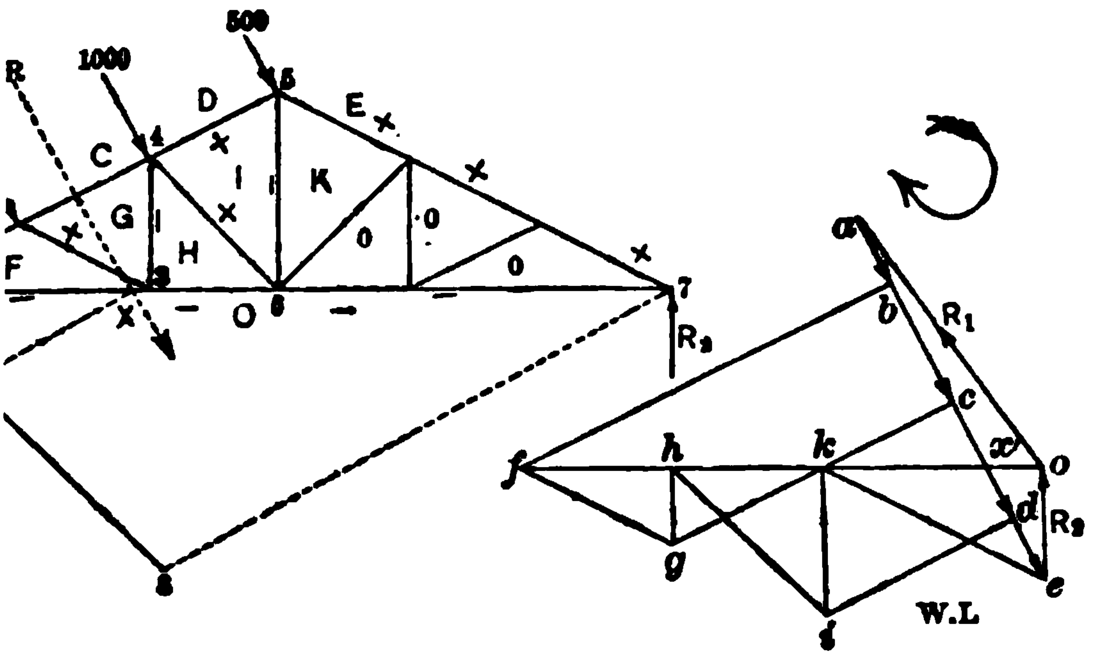


Fig. 49A. Triangular Trus Stress-diagram

the greater reaction being at joint 1. The sum of the reactions are equal to sum of the loads. To find the reactions graphically, draw a line from joint at any angle, say from 30° to 45°, and measure off a distance equal to the t load. In Fig. 49 the line 1–8 represents 3 000 lb. Join 7 and 8, and from draw a line parallel to 7–8, intersecting 1–8 at X'. Then 8–X' is the reactio joint 1 and X'–1 the reaction at joint 7. To draw the stress-diagram, Fig. 4 first draw the load-line *ae* equal to the sum of the loads, in this case 3 000 lb, perpendicular to the rafter 1–5, and divide it so that *ao* is equal to X'–8. Th at joint 1, *oa* is the supporting force, *ab* is 500 lb and *bf* and *fo* are drawn para respectively to *BF* and *FO*, intersecting at *f*. The external forces and stre act in the direction *oa*, *ab*, *bf* and *fo*, showing that *BF* is in compression and in tension. At joint 2 the stress-lines are *fb*, *bc* equal to 1 000 lb, *cg* and *gf*. stress-lines at joint 3 are *of*, *fg*, *gh* and *ho*; at joint 4, *hg*, *gc*, *cd*, *di* and *ih*; a joint 5, *id*, *de*, *ek* and *ki*. If the load-line has been correctly divided at *o*, the stress-lines have been drawn exactly parallel to the lines of the truss, the p *k* will fall vertically above the point *i*. At joint 6 the stress-lines are *ok*, *ki*, *i* *ho*. As the figure must close by a horizontal line through *o*, it is evident that

the truss-diagram cannot be represented, and therefore there can be this member when the wind is from the left. At joint 7 the reaction acting up, and *ok* and *ke* must close the figure, showing that the line s the stress in the entire length of the right rafter, and that there is the bracing on that side of the truss when the wind is from the left. ver, either the lower chord or the rafter is not straight, some of the at side come into action. By noting the character of the stresses in is seen that the different members of the truss have the same kind is produced by vertical loads. As the wind may blow from either is evident that both sides of the truss must be made alike. This trates the method of drawing the stress-diagram for any truss with fter when both ends of the truss are fixed.

Rollers. Example 32. When one end of the truss is FREE TO action at that end must always be practically vertical, and this es a considerable variation of stress when the wind is on different oof; so that it is necessary to draw two wind-stress diagrams, one



Triangular Truss. Truss-diagram and Stress-diagram, Wind Left

THE LEFT, marked W.L, and one for WIND FROM THE RIGHT, It is customary with authors when writing on this subject to he ROLLERS are always under the right-hand support, and this wed here. In practice the ROLLERS may be placed under either es of the truss are usually proportioned to the maximum stresses. le we will take the same truss-diagram that was used in Fig. 49, gain in Fig. 50, which is drawn to show WIND FROM THE LEFT. -line 1–8 and divide it at X', as in example 31. Draw a line *ae*, the rafter and equal to 1–8 in length, and divide it into two same proportions. Through x' on *ae* draw a horizontal line, vertical line, the two intersecting at *o*. Then *eo* represents the at joint 7 and *oa* the reaction at joint 1. The stress-lines at *ab* equal to 500 lb, *bf* and *fo*. At joint 2: *fb*, *bc*, *cg* and *gf*. The diagram W.L. is completed exactly as described for Fig. 49A, ce between the two being the location of point *o*, which gives in the bottom chord for the truss of Fig. 50. Fig. 51 represents ith WIND FROM THE RIGHT. To draw the stress-diagram W.R. pendicular to the rafter and equal to the total load, 3 000 lb. t x' into two segments of the same proportions as the segments

of the line 1–8, Fig. 50, the longer segment being at the top. To find the rea tions draw a horizontal line through x' and a vertical line through t, the two lin intersecting at o. Then do is the reaction at joint 1, and ot the reaction at joi 10. For this diagram it is better to start with joint 10 and take the forces the reverse order from that in which they were taken before. The stress-lin at joint 10 are ol, ts equal to 500 lb, sn and no; at joint 9, ns, sr, rm and at joint 8: on, nm, ml and lo; at joint 7, lm, mr, re, ek and kl; and at joint

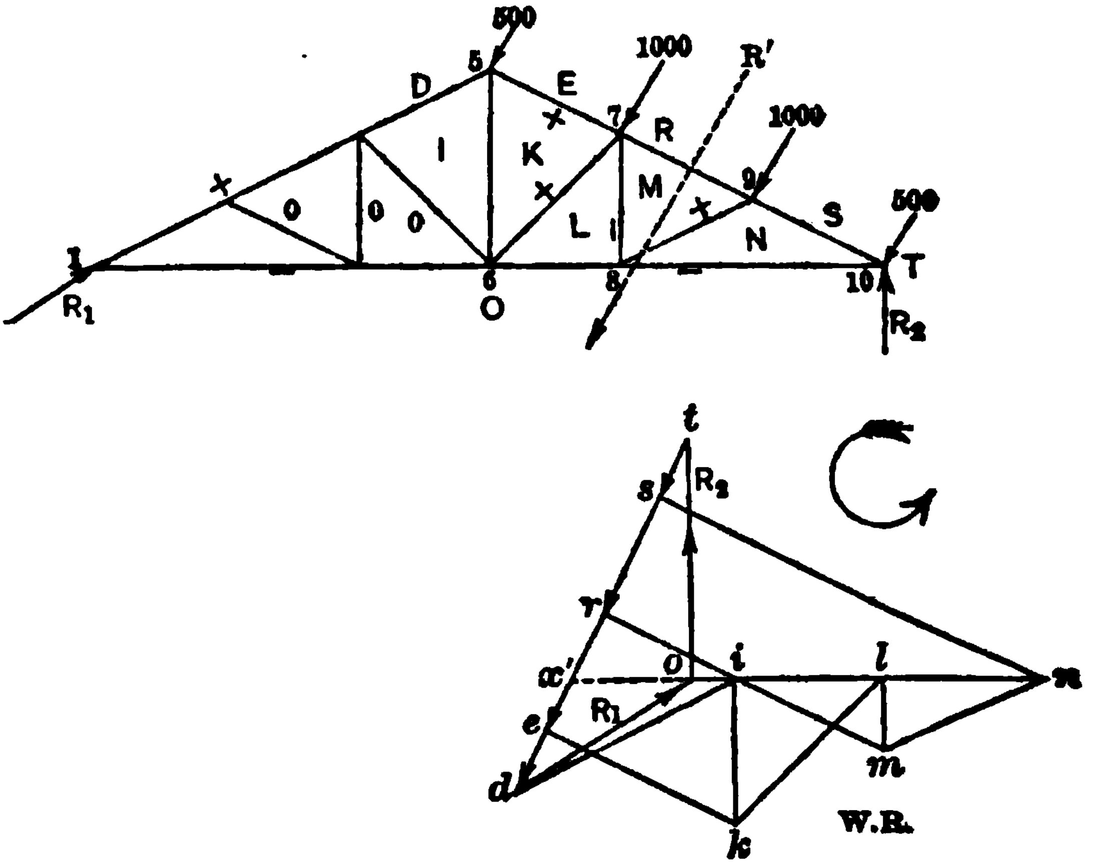


Fig. 51. Triangular Truss. Truss-diagram and Stress-diagram, Wind Right

ke, ed, di and ik. If the diagrams have been correctly drawn the point i fall vertically above the point k. On comparing the two diagrams for W and W.R. it is seen that the stress-lines for the rafters and braces are of the s length and that the stresses are of the same character in both, but that the st in the bottom chord is considerably less when the wind is from the right. T condition does not apply to all trusses, however, so that it is best to draw two stress-diagrams for wind from both directions.

Queen Truss. Example 33. Fig. 52 represents the outline of a QUEEN- TRUSS for a roof having a rise of 14½ in in 12 in. As the truss is of wood supports are considered fixed. Joint 2 divides the rafter into two equal pa consequently the wind-load at this joint is twice that at joint 1 or 4. convenience it is assumed that the wind-load at joint 2 is 1 000 lb and at join and 4, 500 lb. The resultant is 2 000 lb acting through joint 2 and inters the tie-beam at X. To find the supporting forces, draw the line 1–8 equa 2 000 lb and connect 7 and 8. From X draw a line parallel to 7–8 intersec 1–8 at X'. Then 8–X' is R_1 or the supporting force at joint 1 and X'–1 e the supporting force at joint 7. Begin the stress-diagram (Fig. 52A) by dra the line ad at right-angles to the rafter 1–4, and equal in length to 1–8 or 2 oo By means of dividers locate the point o so that oa equals 8–X'. Then the st lines for joint 1 are oa, ab, be and eo; at joint 2, eb, bc, cf and fe; at joint 3 ef, fh and ho; and at joint 4, hf, fc, cd, dk and kh. It is seen that the force-p gon at joint 4 will not close without the brace KH, because the initial poi drawing the polygon is at h, and a horizontal line through d does not

rough h. A QUEEN-ROD TRUSS, therefore, requires braces in the middle panel resist the wind-stress. With the wind from the right, a brace is required from int 3 to joint 6. At joint 5 the stress-lines are oh, hk, kl and lo. It should be ticed that lo acts towards the joint, showing that LO is in compression. At

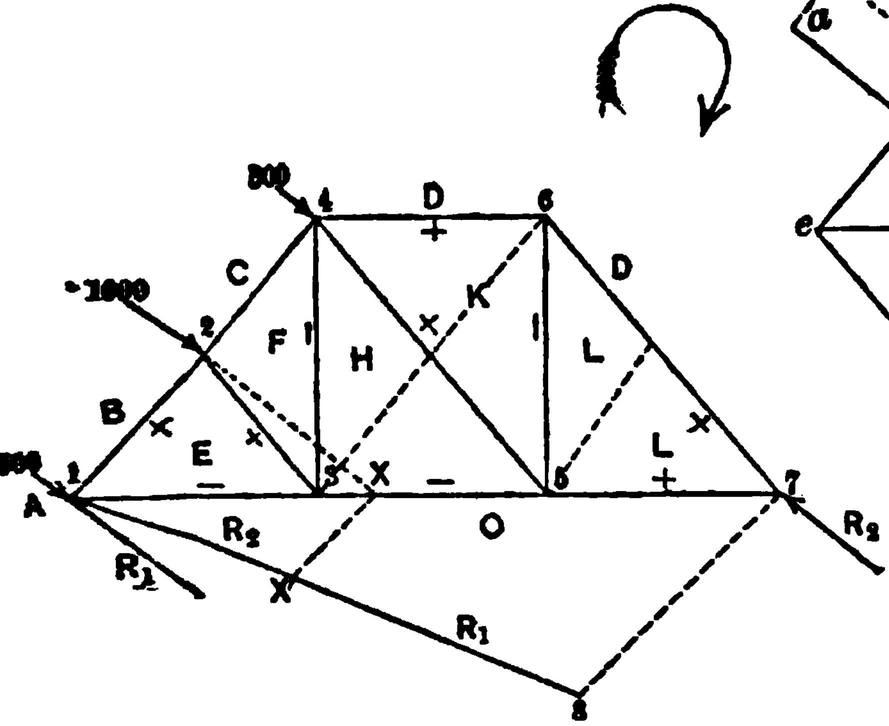


Fig. 52. Queen Truss. Truss-diagram

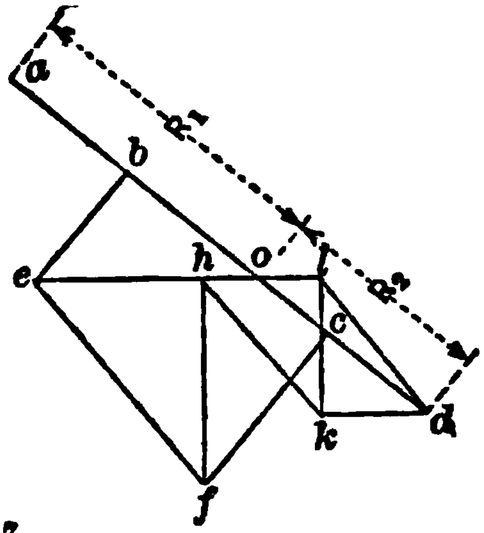
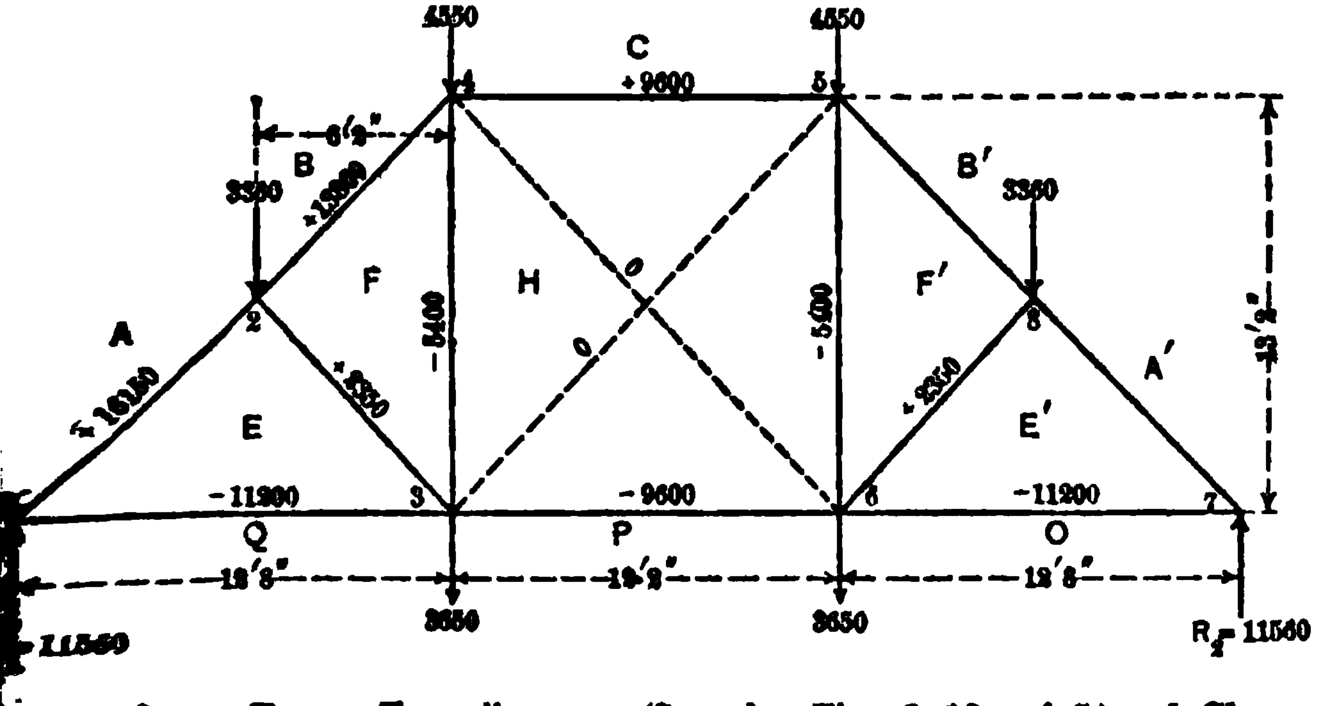


Fig. 52A. Queen Truss. Stress-diagram, Wind Left

t it would seem as though this could not be true, but if we glance at joint 7 see that R_2 is thrusting in on the joint, and that a strut is required to keep joint in position. This condition is true only when the inclination of the ter is greater than 45°. When the inclination of the rafter is exactly 45°,



53. Queen Truss. Truss-diagram. (See, also, Figs. 3, 12 and 54 and Chapter XXVIII, Fig. 1)

e is no stress in LO, and when the inclination is less than 45°, LO is in ten- The stress-lines for joint 6 are lk, kd and dl. If no errors are made, a through d parallel to DL passes through the point l, previously obtained. A slight inaccuracy in locating the point X', or in drawing the stress-diagram,

however, causes the line through d to pass to one side or the other of point and if this happens, it shows that there has been some inaccuracy somewhe In practice, a slight divergence does not materially affect the stress. At join the sides of the stress-polygon are ol, ld and $do = R_2$, the lines being alrea drawn.

Combination of Stresses. Example 34. For the purpose of showing h the stresses due to wind and vertical loads are COMBINED, the truss-diagrams Figs. 53 and 54 are shown, being the same as in Fig. 12, and representing t

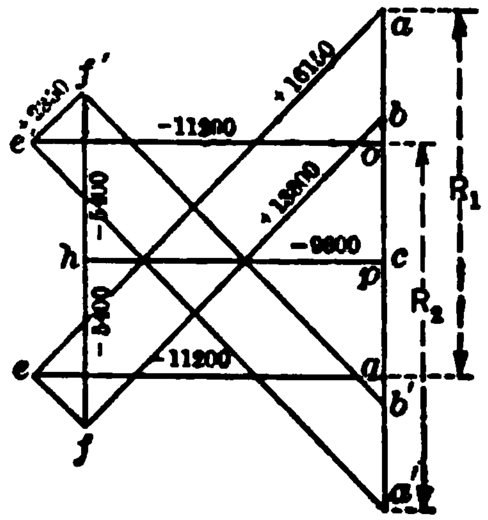


Fig. 53A. Queen Truss. Stress-diagram

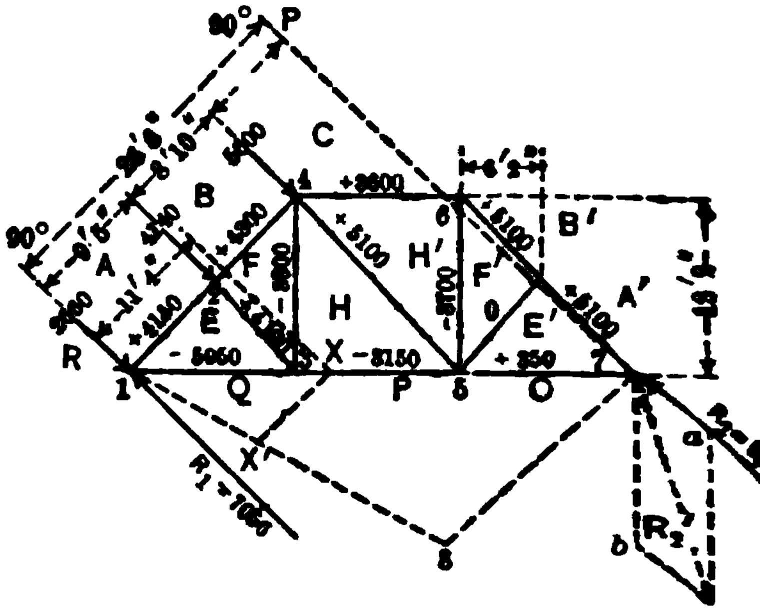


Fig. 54. Queen Truss. Truss-diagram. (See, Figs. 3, 12 and 53 and Chapter XXVIII, Fi

truss shown in Fig. 3. The stresses first determined are those due to the we of the roof and ceiling and to an allowance of 10 lb per sq ft for sleet. page 1055 the roof-area supported at joint 2 was found to be 147½ sq ft an joint 3, 200 sq ft. On page 1055 the weight of the roof was estimated at 12 per sq ft, and allowing 10 lb for sleet, there results 22¾ lb as the greatest load under a heavy wind. This gives 3 360 lb for the load at joint 2 and 4 5 for the load at joint 3. The ceiling-loads will, of course, be the same as in Fi Fig. 53 shows the loads due to weight of materials and sleet, as computed ab and the ceiling-loads. Fig. 53A is the stress-diagram for these loads, with stresses indicated by figures. This diagram is drawn exactly in the same as the stress-diagram in Fig. 12, page 1071.

Wind-Stresses. The inclination of the roof is very close to 45°, and from T IX, page 1053, the normal wind-pressure for that angle is found to be 28 lb. M plying the roof-area at joints 2 and 3 by 28, the wind-loads indicated in Fi are obtained. The wind-load at joint 1, also, must be found. The roo supported at this joint, allowing 17 in for eave-projection (Fig. 3) is 6⅓ by or 95 sq ft, which makes the wind-load 2 660 lb. The next step is to fin point at which the resultant of these loads cuts the rafter. As the load not symmetrical or uniform on the rafter, the point through which the res acts must be determined by means of moments about joint 1. The arms loads at joints 2 and 4 are figured on the truss-diagram (Fig. 54). The mo are

$$4\,140 \text{ lb} \times 9\tfrac{5}{12} \text{ ft} = 38\,985 \text{ ft-lb}$$
$$5\,600 \text{ lb} \times 18\tfrac{1}{4} \text{ ft} = 102\,200 \text{ ft-lb}$$
$$\text{The sum of the moments} = 141\,185 \text{ ft-lb}$$

it is the sum of all the loads, or 12 400 lb, and the distance of its point n from 1 is found by dividing the sum of the moments by the result- 141 185 ft-lb divided by 12 400 lb = 11.4 ft. Measuring off 11.4 ft from joint 1 and drawing a line at right-angles to it intersecting the e point X is determined. From 1 the line 1–8 is drawn at any ual in length to the sum of the loads, 12 400 lb, and 7–8 is drawn. ne is drawn parallel to 7–8, intersecting 1–8 at X'. Then 8–X' is porting force at joint 1 and X'–1 is R_2 or the supporting force at

Forces Computed by Moments. The supporting forces may also by moments. The moments of the loads about joint 1 tend to ss from left to right. To prevent this rotation there is the moment ting force R_2 acting at joint 7 to rotate the truss from right to left. equilibrium, the moment of R_2 about joint 1 must just equal the sum ts of the loads about the same point. This sum was found above to lb. The arm of R_2 is the perpendicular distance between its line l joint 1. Continue R_2 to meet the dotted line at P. The dis- ce RA to P scales, say 26.5 ft. (By trigonometry, 26 ft.) Knowing alue of R_2 is obtained by dividing the oments of the loads, 141 185, by the t. This gives 5 344 lb. As the sum must equal the total load, R_1 equals 344 lb, or 7 056 lb. The distance should scale reasonably close to these ring the supporting forces, the stress- 54A, is drawn exactly as described for the inclination of the rafters is a han 45°, OE' is in compression, but ery small. The figures on Fig. 54A resses in pounds. The stresses may ited and should be arranged as in the following table. In wind-stresses, it should be remembered that the wind may blow de of the truss, and the greatest stress liable to occur should be

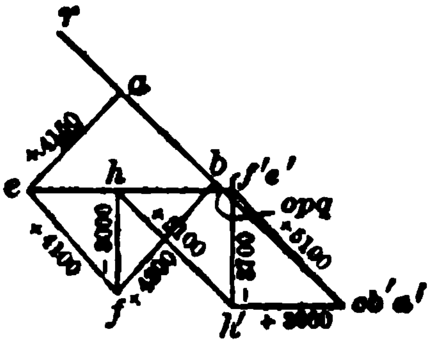


Fig. 54A. Queen Truss. Stress-diagram, Wind Left

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II. Stresses for the Trusses Shown in Figs. 12, 53 and 54

	Dead weights and sleet (Fig. 53A)	Wind-stresses (Fig. 54)	Totals	Stresses (Fig. 12)
.	+16 150	+5 100	+21 250	+25 600
.	+13 800	+5 100	+18 900	+21 300
.	+ 9 600	+3 600	+13 200	+14 700
.	+ 2 350	+4 100	+ 6 450	+ 4 400
.	0	+5 100	+ 5 100	0
.	− 5 400	−3 700	− 9 100	− 6 900
.	−11 200	−5 950	−17 150	−17 600
.	− 9 600	−3 150	−12 750	−14 700

rs are lettered as in Fig. 54. Thus the stress in the rafter $F'B'$ the rafter on the other side, and this stress acts through the he rafter; hence the stress for AE and BF should be entered tress in $F'B'$. In the same way the stress in the rod $H'F'$ is

greater than in FH; hence the stress in $H'F'$ should be tabulated. The stre in OE' slightly reduces the tension due to the dead load, but as the stress in E increases it, the stresses in EQ and HP should be tabulated. Both sides of t truss should of course be made alike, and two braces should be inserted in t middle panel. In the fifth column of the table are given the stresses due to t ceiling-load and a vertical load on the roof of 42¾ lb per sq ft, as obtained fro the stress-diagram, Fig. 12. Comparing the stresses in the fourth and fif columns, it is seen that except for the brace EF, and for the two rods, the stres obtained by combining snow and wind and adding to the dead weight are great than the totals due to wind, dead weight and sleet. Vertical loads, of cour cause no stresses in the braces of the middle panel, and unless the wind-stres are drawn, it is necessary to estimate the sizes of these braces. The stresses these braces, however, are so small that large pieces of timber are not requir The stresses given in the fourth column are unquestionably nearer what t real stresses are likely to be than those in the fifth column. If the roof is erect in a warm climate where there is no sleet, these stresses may be further reduc by omitting the 10 lb per sq ft added for sleet. If, on the other hand, the clination of the roof is less than 30°, the stresses produced by a heavy fall snow without wind generally exceed the sum of those due to dead weight, sl and wind; and for such roofs the stresses due to the maximum snow-load sho always be computed.

Reactions. The reactions, or supporting forces of the truss shown in Fig. are very much inclined from the vertical. As the dead load, however, is alwa acting on the truss, the inclination of the real reaction is never so great, more nearly vertical; and when there is no wind the reactions are exactly v tical. The theoretical reaction, due to both wind-load and dead load, is diagonal of a parallelogram, the two adjacent sides of which are the reacti for the dead load and wind-load drawn to the same scale. Thus if a–7, Fig. represents the reaction due to the wind and b–7 the vertical reaction, due to dead load and drawn to the same scale, then R'_2 is the resultant reaction, modif somewhat, however, by friction. Examples 31, 32 and 33 serve to show general method of drawing WIND STRESS-DIAGRAMS, and are sufficient to ena the student to draw those diagrams for most trusses with straight rafters. trusses with curved rafters the diagrams become more complicated, and reader is referred to Graphical Analysis of Roof Trusses, by Charles E. Gre and to other standard handbooks on the subject.

7. Trusses with Knee-Braces

Knee-Braces are generally used to give greater stability to the structure whole when roof-trusses are supported by COLUMNS. Under the action vertical loads the stresses in these members are usually assumed as zero, wh would be true if the materials composing the TRUSS, KNEE-BRACES and COLU were rigid. This discussion will deal, however, with the effect of wind blo against one side of the building and roof. The ACTUAL STRESSES in the k braces, columns and truss-members will probably never be known exactly there are so many VARIABLE FACTORS entering into the problem. In the u construction, in which columns are bolted to masonry pedestals at the bott either riveted or bolted to the trusses at the top, and in which the knee-br are riveted at both ends, the degree to which these connections may be consid FIXED is a question leading to many arguments and differences of opinion. will not be discussed at all; but it will be shown how the stresses in all mem of the framework can be found under given assumptions. Assume, for exam that the bottoms of the columns are sufficiently FIXED, so that a point of

midway between the bottom of the knee-brace and the masonry
uivalent to assuming a PIN at this point), and so that the top attach-
those of the knee-braces may be considered as PIN-CONNECTIONS.
russ and loading shown in Fig. 55, it is clear that the outside forces
equilibrium, and, unless the points M and N are unlike in some

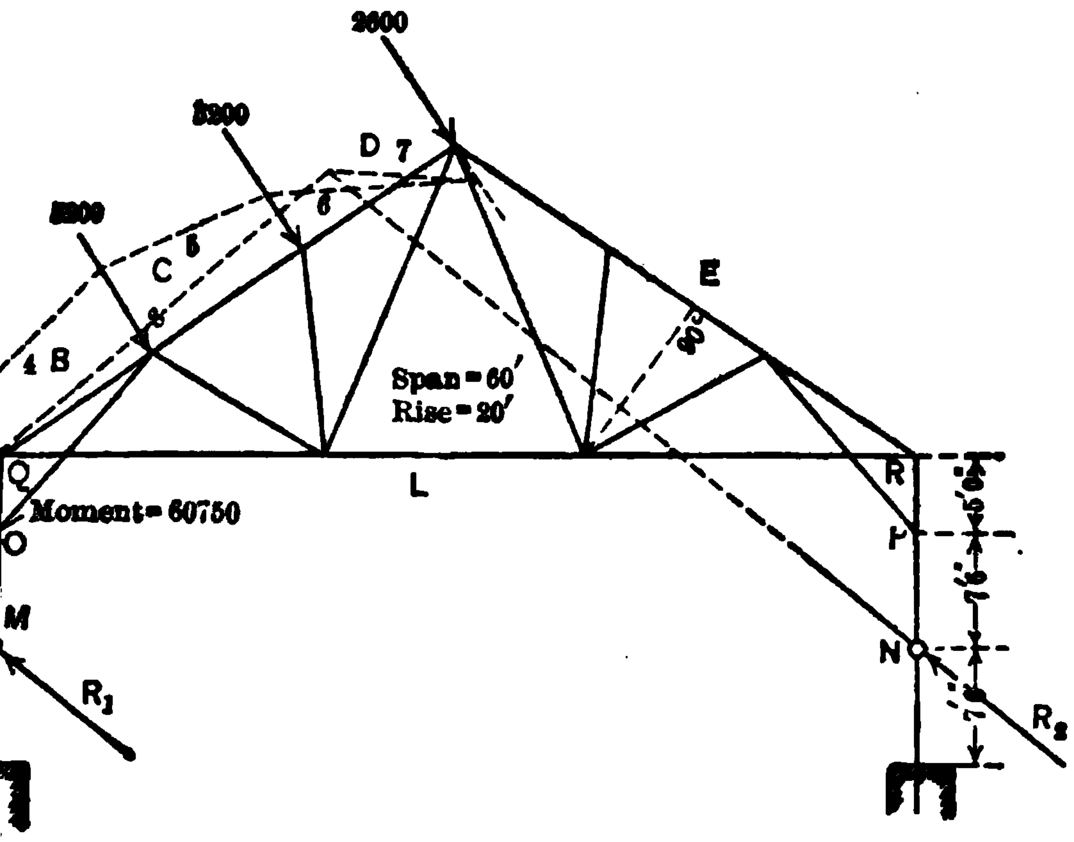


Fig. 55. Truss with Knee-braces. Truss-diagram

reactions at these points will be parallel to the direction of the
e wind-forces. Lay off to any convenient scale the wind-forces
own in Fig. 55A. Then XY is the direction and magnitude of the
-pressure and also the direction of R_1 and R_2. The magnitudes
found by means of the equilibrium polygon explained on page 1097.
SX and R_2 to YS. These
orrect in direction and mag-
ome condition is imposed to
If there are no moments at
hese points are RESTRAINED
ertically, the vertical com-
nd R_2 must remain constant,
treme case where M may be
PIN-CONNECTION and N as
ERS. Any assumption may be
magnitudes of the horizontal
these points as long as the
wo equals the sum of the
onents of R_1 and R_2. It is
ssume these as equal. In

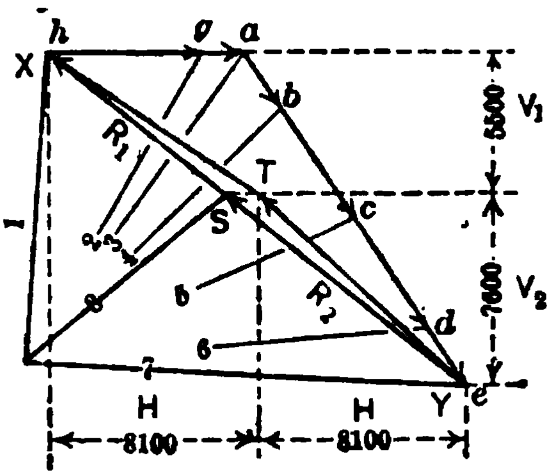


Fig. 55A. Truss with Knee-braces. Force-polygon

ctions at M and N are TX and YT, respectively. The next step
ect of these reactions at the points O, Q, P and R. The vertical
and V_2 act as vertical forces at O and P. The horizontal com-
bending moments at O and P, and, in effect, horizontal forces
R. Taking the left column, the 8 100 lb acting towards the left
column to the left if not prevented by the joints at O and Q.

If the member MQ is considered to act as a lever with a fulcrum at O, a hor zontal force of 8 100 lb acting towards the left at M will produce a pressure, or force acting from left to right at Q which equals, by the method of moments, th center of moments being at O, 8 100 lb × 7.5 ft ÷ 5 ft = 12 150 lb. At O, in li manner, taking the center of moments at Q, 8 100 lb × 12.5 ft ÷ 5 ft = 20 250

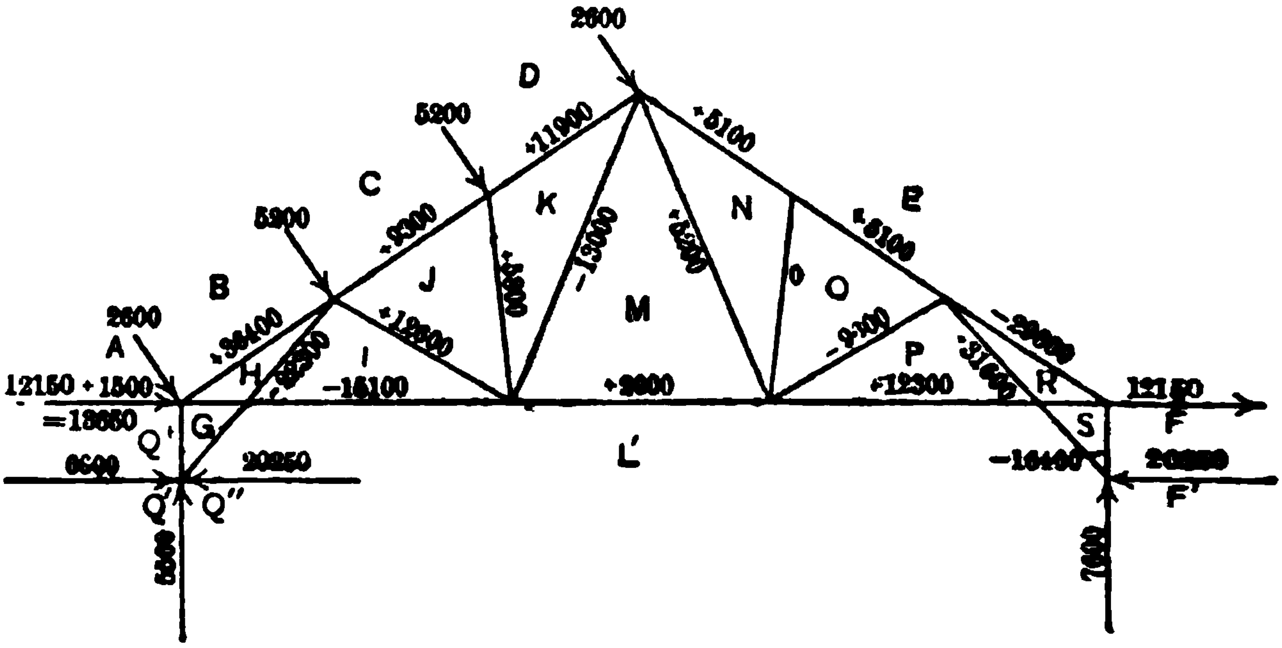


Fig. 56. Truss with Knee-braces. Truss-diagram

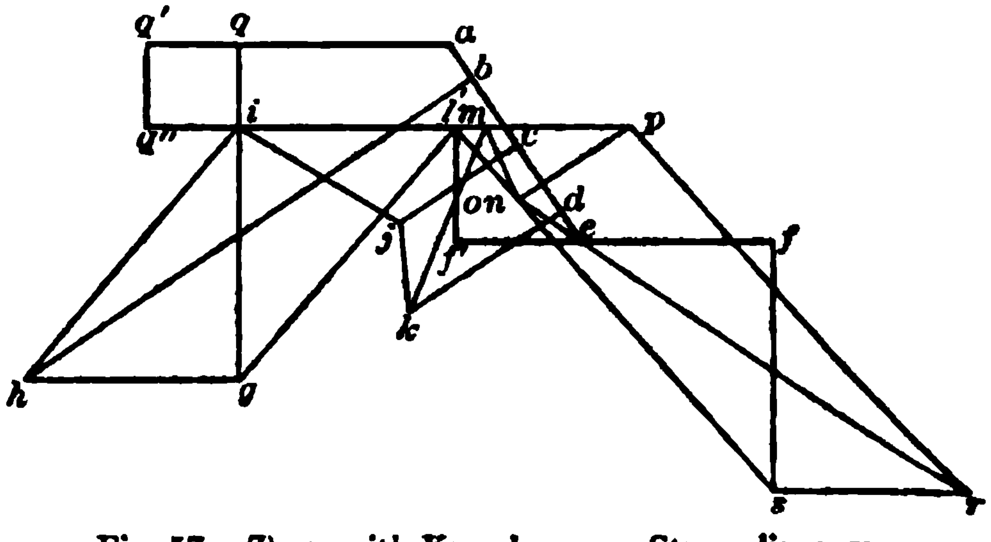


Fig. 57. Truss with Knee-braces. Stress-diagram

is produced, acting from right to left. These forces are shown in Fig. 56. Wh combined with those shown in Fig. 55 they give the forces acting at O, Q and P which are used in constructing the stress-diagram shown in Fig. 57.

8. Arched Trusses

An Arched Truss is one which has the FORM OF AN ARCH and which is supported at the ends that the reactions produced by vertical forces are verti This is usually accomplished by placing PIN-CONNECTIONS at the supports a providing ROLLERS at one end to permit horizontal movement.

Stresses in an Arched Truss. The determination of the stresses in members of an ARCHED TRUSS is readily accomplished by following the meth given in the previous examples.

Arched Truss with Roller-Support. Example 35. In Fig. 58 is sh the left half of an ARCHED TRUSS and the ROLLER-SUPPORT. This truss has shape and dimensions of a truss in the Live Stock Pavilion, Union Stock-Y

and Transit Company, Chicago, Ill. It is discussed in the Engineering News of June 28, 1906. The loading shown is symmetrical about the middle of the span and hence each reaction equals one-half the total load. Fig. 58A shows

Fig. 58. Live Stock Pavilion, Chicago, Ill. Truss-diagram

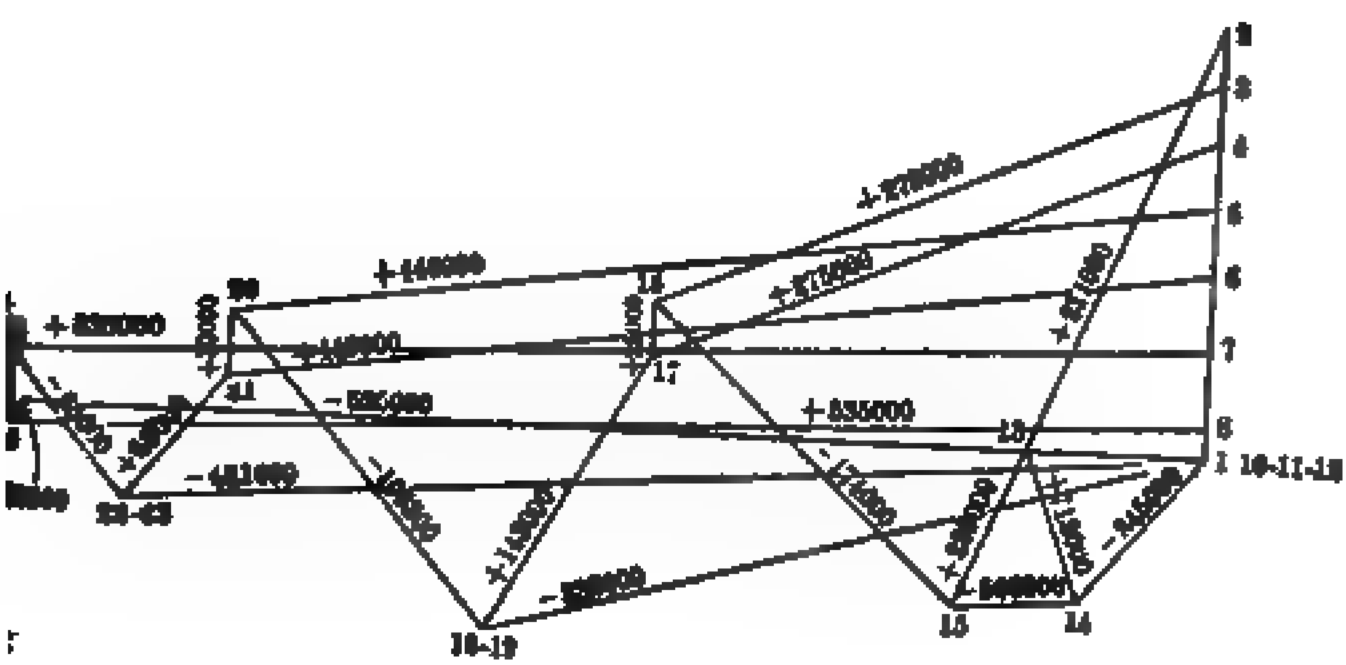


Fig. 58A. Live Stock Pavilion, Chicago, Ill. Stress-diagram for Truss

stress-diagram for one-half of the truss. The stresses upon the right of the ... are the same as those upon the left.

... HORIZONTAL DEFLECTION of this truss is measured by the movement of ...ROLLER-END. This movement is computed in the manner explained for the ... TRUSS, pages 1085–7, by the formula $D = \Sigma(Sul \div AE)$. Where D is the

HORIZONTAL MOVEMENT, S the stress in any member as given by the stress-diagra shown in Fig 58A, u the stress in any member produced by the unit load appli at the roller end of the truss and acting in a horizontal direction (Fig. 58B l the length of any member, A the area of any member, E Young's modulus elasticity for the material composing any member and Σ the sign of summatio and when limits are not designated, the formula indicates that $\Sigma(Sul \div AE)$ is be taken for each member of the truss. For the loads and areas indicated Fig. 58, the rollers will move about 10 in wh E is 30 000 000 lb per sq in. In order that given span may obtain under a given lo each tension-member must be construct shorter than its geometrical length by amount which it is lengthened by the str which it resists, and each compression-memb must be lengthened in a like manner. A other loading will produce a change in length of the span. To reduce the HORIZONT DEFLECTION without changing the lengths the members they would have to be made excessively heavy. A truss of t form shown in Fig. 58 is not economical as an ARCHED TRUSS on rollers b may be satisfactorily used by connecting the two end-pins by a TIE-ROD.

Fig. 58B. Live Stock Pavilion, Chicago, Ill. Stress-diagram

Arched Truss with Tie-Rod. When a TIE-ROD is employed the memb become much lighter and can be built according to their geometrical lengt The stress in the TIE-ROD may be found from the formula

$$S_t = \sum \frac{Sul}{AE} \div \left\{ \sum \frac{u^2l}{AE} + \frac{l'}{A'E'} \right\}$$

in which S_t is the stress in the tie-rod, A' the area of the tie-rod, l' the length the tie-rod and E' Young's modulus of elasticity for the material compos the tie-rod. The other symbols have the significance given above for expression for D. Since the stress and area of the tie-rod appear in the ab equation it is necessary to assume an area and then compute the value of If this produces a unit stress in the tie-rod differing greatly from the allowa value, a new trial must be made. Having found the stress in the tie-rod, resulting stresses in the truss-members can be found graphically from a str diagram which will be of the form shown in Fig. 58B, which was constructed a horizontal force of 1 000 lb. The stresses can be found, also, by multiply the stresses produced by one pound by the value of S_t. The stresses produ by S_t combined algebraically with those obtained from Fig. 58A give the fi stresses. These stresses differ but little from those which obtain for a T HINGED ARCH of the form shown in Fig. 58, and such structures with the t ROD are often classed as TWO-HINGED ARCHES

Assumption of Areas. Since the DEFLECTION of the truss shown in Fig. depends upon the AREAS of the members, it is evident that they must be ei known or assumed before the formulas for D or S_t can be applied. For a structure the AREAS are of course unknown and the problem of determining stresses becomes one which is sometimes classed as CUT-AND-TRY. For the trial, the areas may be assumed as unity and the corresponding value o found and then the combined stresses. The members may now be designe to area and a new trial made with these areas. Usually the second tri sufficient, as a slight change in areas does not materially affect the va of S_t.

9. Trussed Arches

Symmetrical Trussed Arches. The THREE-HINGED ARCH is the simplest form of TRUSSED ARCH, and, as used in buildings, it is usually symmetrical in form, consisting of two trusses connected by a pin over the middle of the span and resting on a pin at each support. The stresses in the truss-members are found by the ordinary graphical methods after the reactions have been determined.

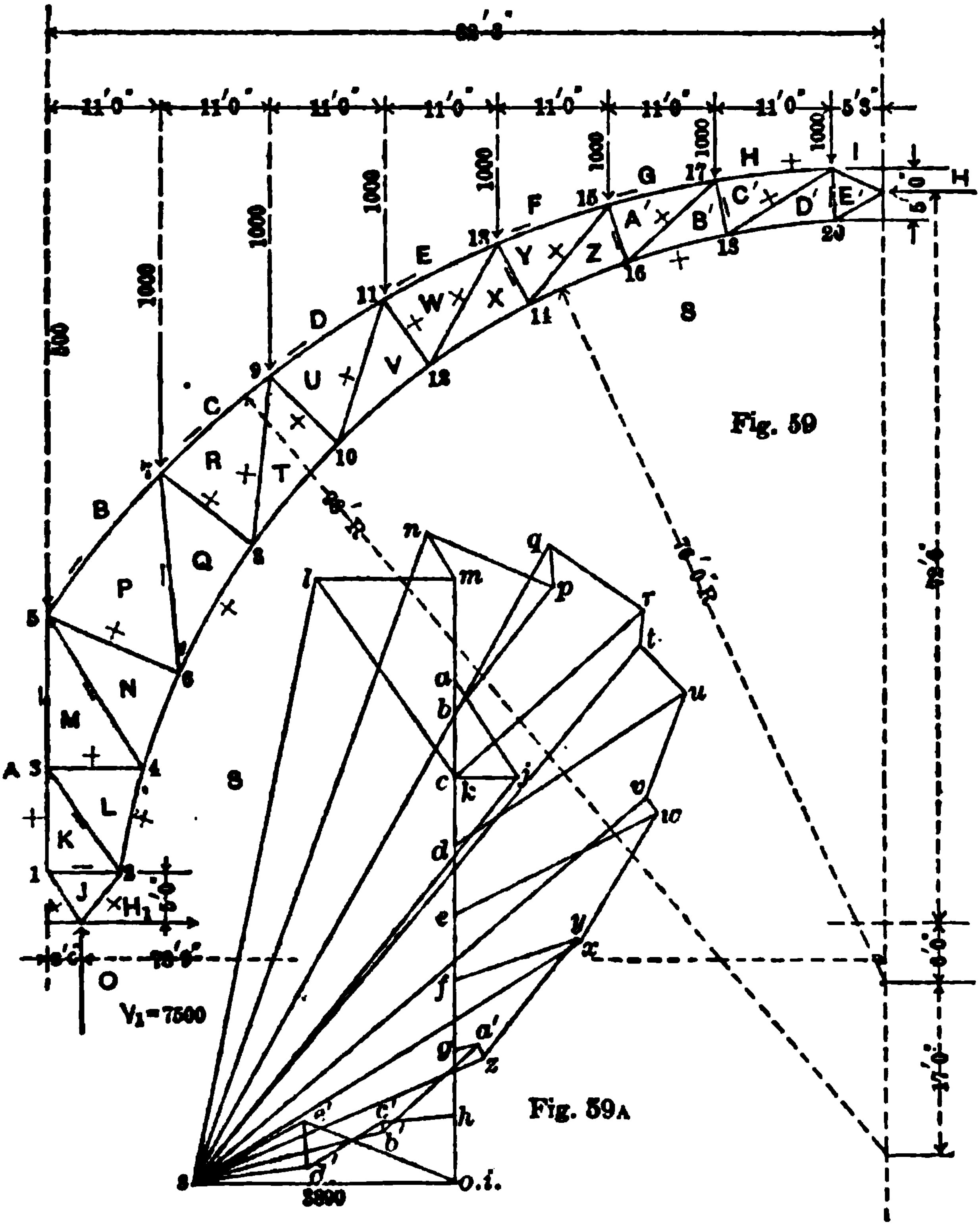


Fig. 59. Three-hinged Arch. Truss-diagram
Fig. 59A. Stress-diagram

The SUPPORTING FORCES are inclined and may be resolved into two components, one vertical and the other horizontal. For symmetrical loading the two reactions are equal in magnitude. The vertical components are each equal to one-half the vertical loading. The horizontal components are equal in magnitude and opposite in character. The following examples illustrate the methods to be followed in the determination of the stresses.

Trussed Three-hinged Arch. **Example 36.** Fig. 59 shows one-half o TRUSSED THREE-HINGED ARCH with a vertical load of 1 000 lb per top-ch joint. Fig. 59A shows the stress-diagram for this loading; but before it can drawn, the vertical and horizontal reactions at the left support must be det mined. The vertical reaction is $(7 \times 1\,000) + 500 = 7\,500$ lb or one-half the v tical load. The horizontal component or the HORIZONTAL THRUST of the a may be found by moments. The center of moments will be taken at the mid

Fig. 60. Liberal Arts Building, Chicago, Ill. Truss-diagram
Fig. 60A. Stress-diagram

pin at the crown as at this point the moment is zero. The equation of mome is $H_1 \times 72.5 + 1\,000\,(5.25 + 16.25 + 27.25 + 38.25 + 49.25 + 60.25 + 71.25)$

$$+ 500 \times 82.25 - 7\,500 \times 78.75 = 0,$$

or

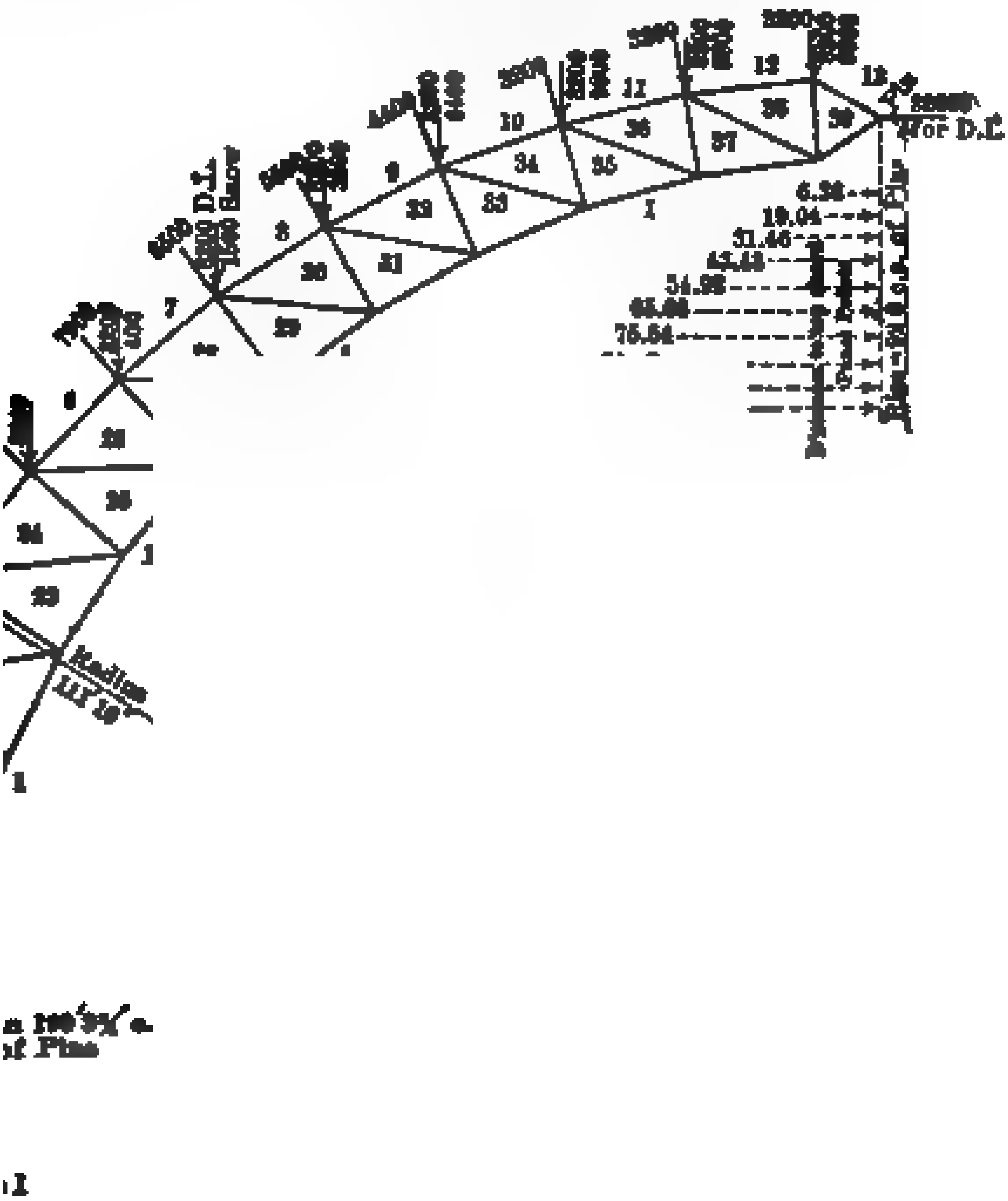
$$H_1 = 281\,750 \div 72.5 = 3\,886 \text{ lb}$$

Having determined V_1 and H_1, the stress-diagram shown in Fig. 59A ca readily constructed. Since the arch is symmetrical, it is necessary to draw one-half the stress-diagram. If the right half of the arch is removed and i place a horizontal force applied at the middle pin, the magnitude of this forc

: HORIZONTAL THRUST H_1, since, for equilibrium, the algebraic sum
zontal forces is zero.

Three-hinged Arch. **Example** 37. Fig. 60 represents one-half of
THREE-HINGED ARCH used in the Liberal Arts Building of the Colum-
tion, Chicago, Ill., 1893. (See Engineering Record, July 9, 1892.)
the stress-diagram for the loading shown in Fig. 60.

tion of Stresses. In Examples 35 and 36 only the effect of vertical
een considered. Where THREE-HINGED ARCHES are employed they
igned to carry dead, snow and wind-loads. The dead and snow-



61. 5th Regiment Armory, Baltimore, Md. Truss-diagram
Fig. 61A. Stress-diagram

al loads but the snow-load is not symmetrical in all cases. The
ually considered as acting normal to the roof. In order to be
aximum stresses are obtained, the stresses for the following com-
g must be found and combined

AD LOAD only,
OW-LOAD covering left half of roof,
OW-LOAD covering right half of roof,
ND-LOAD acting normal to roof on left of center,
ND-LOAD acting normal to roof on right of center.

Fig. 61B. 5th Regiment Armory, Baltimore, Md. Stress-diagram

Fig. 61C. 5th Regiment Armory, Baltimore, Md. Stress-diagram

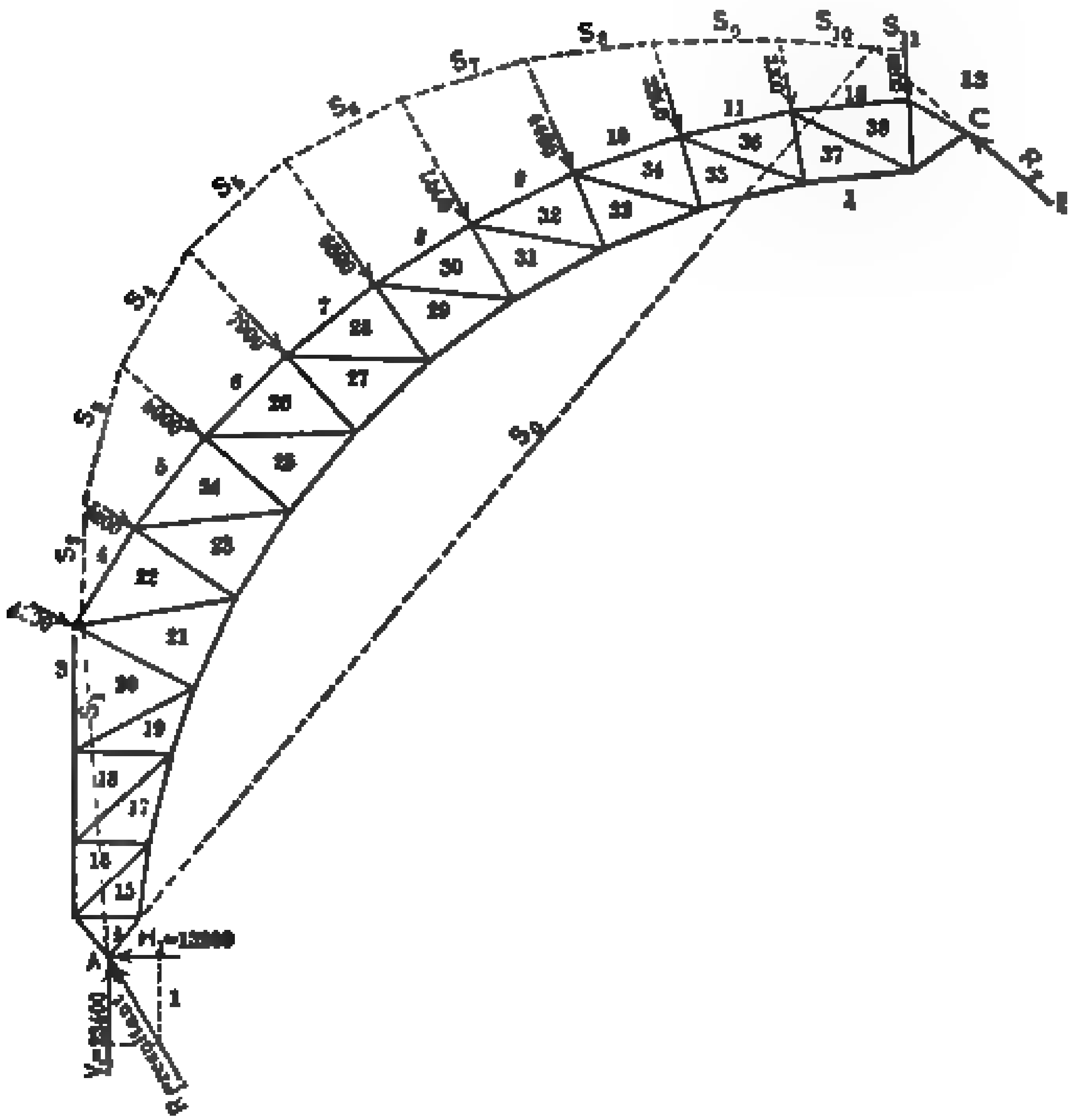


Fig. 61D. 5th Regiment Armory, Baltimore, Md. Truss-diagram

esses for the above conditions of loading are to be found for one-half h. In combining the stresses those which occur at the same time are d in determining maximums. Many engineers do not consider snow loads acting on the same portion of the roof simultaneously.

d **Three-hinged Arch.** **Example 38.** Fig. 61 shows one-half of a HREE-HINGED ARCH with the dead, snow and wind-loads indicated at e upper-chord joints. This form of truss supports the roof of the nent Armory, Baltimore, Md., described in the Engineering Record , 1904. The stresses for the loadings specified above will be deter- it will be shown how these are to be combined.

ad Stresses. The reactions are obtained by the method used in 5. V_1 is 77 900 lb and H_1 32 000 lb. Fig. 61A is the stress-diagram nbers shown in Fig. 61.

Left Half of Span. Assuming that the snow covers the portion of own in Fig. 61 and taking the center of moments at the middle pin, y moments that V_1 is equal to 26 700 lb and H_1 is equal to 15 000 lb. t the support the stress-diagram shown in Fig. 61B is readily drawn.

Right Half of Span. With the snow on the right of the crown, the the span shown in Fig. 61 is unloaded. The total snow-load is nd it has just been found that the vertical reaction at the support the loading is 26 700 lb; hence the vertical reaction at the other 1 200 less 26 700 lb or 14 500 lb or V_1 for the case considered. Since at the middle pin is zero, V_1 (half the span) less H_1 (rise of the arch) or $14\,500 \times 95.16 - H_1 \times 92.0 = 0$; and $\times 95.16) \div 92 = 15\,000$ lb which is the nd above. As before, beginning at the , the stress-diagram is constructed as . 61C.

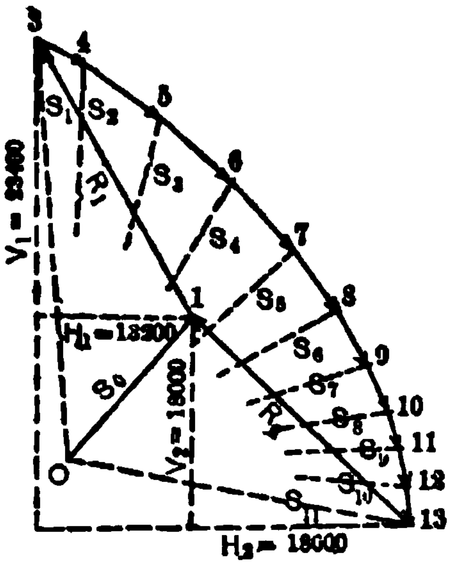


Fig. 61E. 5th Regiment Armory, Baltimore, Md. Force polygon

ering Entire Span. The algebraic sum es found from the two cases above for vill give the stresses produced by a vering the entire span.

on Left of Crown. Here no two of parallel. This condition increases the ng the reactions. These may be com- oments, but a graphical method is onvenient. The direction and mag- resultant of the wind-forces are first hics. As shown in Fig. 61E, the wind- off in order. Then 3–13 is the direc- itude of the resultant. Next, from any point O draw the strings and construct the equilibrium polygon shown in Fig. 61D, begin- ing string S_1 from A, and so on until string S_{11} cuts the line BC gh the middle pin and the pin at the right support. This is the e reaction at the right support. In Fig. 61E, from 13 draw a line and from O a line parallel to S_0 in Fig. 61D, and prolong them until . Then 1–3 is the reaction at A and 13–1 that at the right support. se into vertical and horizontal components, V_1 equals 23 400 lb, 00 lb, V_2 equals 18 000 lb and H_2 equals 18 600 lb. Fig. 61F shows ram from the left support up to the crown.

on Right of Crown. Since the reaction at A, Fig. 61D, produced ust pass through the hinges, or pins A and C, the stress-diagram

will be exactly similar in shape to that shown in Fig. 61C; but the values of I and H_2 will be 18 000 lb and 18 600 lb respectively. The stresses will bear direct proportion to the stresses found from Fig. 61C, and hence a new diagra is not necessary.

20 21

Fig. 61F. 5th Regiment Armory, Baltimore, Md. Stress-diagram

Combination of Stresses. The maximum stresses may now be determine To illustrate the method, consider the lower chord 1–37.

	lb
(*a*) DEAD-LOAD stress,	+ 22 100
(*b*) SNOW on left of crown,	− 14 300
(*c*) SNOW on right of crown,	+ 37 800
(*d*) WIND-LOAD on left of crown,	− 31 600
(*e*) WIND-LOAD on right of crown,	+ 46 900
(*f*) SNOW over all,	+ 23 500
Total stress without wind,	+ 59 900
(*a*) + (*e*),	+ 69 000
(*a*) + (*d*),	− 9 500

The maximum stresses are 69 000 lb compression and 9 500 lb tension, assu ing that the wind and snow-loads are not considered to act on the same side the crown. If no such restriction is made, the maximum stresses are 106 800 compression and 23 800 lb tension. In a like manner the maximum stress each member of the truss is determined. Tables XIX and XX give the MA IMUM STRESS for the members shown in Fig. 61.

Stress-Diagrams for Three-hinged Arches. The STRESS-DIAGRAMS in t above cases are very difficult to construct owing to the great number of li and the difficulty in drawing them exactly parallel to the lines of the tru diagram. One or more members should be computed as a check on the grap ical work

Three-hinged Arch with Tie-Rod. The introduction of a TIE-ROD conne ing the end-pins of a THREE-HINGED ARCH and placing ROLLERS under one e

changes the ARCH into a SIMPLE TRUSS composed of three members,
l rafters and a horizontal tie. Under vertical loading, the support-
ire vertical, but for wind-loads the supporting force at the end with-
is inclined. The stresses in the truss-members are the same as found
he THREE-HINGED ARCH. The stress in the tie-rod equals the hori-
st found above at the roller-end for the given loading. The support
-end is designed for vertical forces only, while the support at the other
sist the vertical reaction and the total horizontal component of the
g on the structure, or for roofs the horizontal component of the wind-
is is very much smaller than the horizontal force which must be
n the structure is without a tie-rod or a true THREE-HINGED ARCH.

Table XIX. Three-hinged Arch. Chord-Stresses

Thousand pounds

Dead load, ig. 61A	Snow on left of crown, Fig. 61B	Snow on right of crown, Fig. 61C	Snow over all the roof	Wind on left of crown, Fig. 61F	Wind on right of crown.* Fig. 61G	Max. stresses	
						Tension	Compression
· 25.2	+ 6.1	− 2.1	+ 4.0	+25.6	− 2.6	....	50.8
· 6.3	− 7.0	−14.0	− 21.0	+32.8	−17.4	30.7	26.5
· 18.7	−13.2	−21.4	− 34.6	+42.6	−26.5	58.4	23.9
· 19.0	−13.9	−22.8	− 36.7	+45.8	−28.3	61.2	26.8
· 22.4	−15.5	−28.0	− 43.5	+58.7	−34.7	72.6	36.3
26.8	−15.8	−35.1	− 50.9	+73.8	−43.5	86.1	47.0
26.6	−11.2	−41.0	− 42.2	+85.3	−50.8	88.6	58.7
22.0	− 3.9	−44.6	− 48.5	+90.7	−55.3	81.2	68.7
13.3	+ 7.8	−44.8	− 37.0	+89.6	−55.6	68.9	76.3
3.7	+20.2	−41.7	− 21.5	+81.9	−51.7	55.4	78.2
6.4	+30.0	−33.5	− 3.5	+67.2	−41.5	35.1	73.6
14.3	+30.0	−20.6	+ 9.4	+47.2	−25.5	11.2	61.5
18.7	+19.3	− 2.8	+ 16.5	+23.3	− 3.5	....	42.0
21.8	+22.6	− 3.3	+ 19.3	+27.8	− 4.1	....	49.6
18.8	− 5.4	+21.9	+ 16.5	− 6.7	+27.2	....	46.0
22.1	−14.3	+37.8	+ 23.5	−31.6	+46.9	9.5	69.0
33.3	−11.3	+51.9	+ 40.6	−53.2	+66.0	19.9	99.3
47.7	+ 2.1	+61.0	+ 63.1	−69.5	+75.6	21.8	125.4
63.0	+18.1	+64.8	+ 82.9	−78.3	+80.4	15.3	161.5
18.4	+31.5	+65.1	+ 96.6	−80.1	+80.7	61.7	130.6
90.3	+41.1	+61.8	+102.9	−75.0	+76.6	....	208.0
98.7	+45.7	+55.8	+101.5	−62.8	+69.2	....	213.6
02.6	+46.0	+48.5	+ 94.5	−46.3	+60.1	....	208.7
01.8	+42.9	+40.0	+ 82.9	−26.0	+49.6	....	194.3
02.3	+42.3	+37.9	+ 80.2	−20.3	+47.0	....	191.6
86.7	+34.8	+29.3	+ 67.7	− 9.6	+36.3	....	157.8
59.5	+22.4	+16.2	+ 38.6	+ 3.9	+20.1	....	102.0
77.4	+29.2	+21.0	+ 50.2	+ 5.2	+26.0	....	132.6

* By proportion, 18 600 : 15 000.

ad no Rollers. If the ROLLERS are omitted and a TIE-ROD is used,
e tie-rod and the reactions are indeterminate. They depend upon
gidities of the tie-rod and the material composing the supports.
is made very heavy so that its stretch will be very small when

stressed, the stresses in all members of the structure may be taken the same found for the condition where rollers are used, and the horizontal component the wind-load equally divided between the supports.

Table XX. Three-hinged Arch. Web-Stresses

Thousand pounds

Member, Fig. 61	Dead load, Fig. 61A	Snow on left of crown, Fig. 61B	Snow on right of crown, Fig. 61C	Snow over all the roof	Wind on left of crown, Fig. 61F	Wind on right of crown,* Fig. 61C	Max. stresses Tension	Max. stresses Compressio
39–38	− 4.9	− 3.8	+ 2.1	− 1.7	−13.7	+ 2.6	16.5	....
36–37	+10.4	+ 0.3	+14.5	+14.8	−18.5	+18.0	8.1	28.7
34–35	+14.5	+ 8.3	+13.0	+21.3	−18.6	+16.1	4.1	38.9
32–33	+17.8	+12.7	+10.6	+23.3	−15.4	+13.1	....	43.6
30–31	+19.7	+13.8	+ 7.6	+21.4	−10.7	+ 9.4	....	42.9
28–29	+20.3	+12.3	+ 4.7	+17.0	− 4.9	+ 5.8	....	38.4
26–27	+18.7	+ 9.3	+ 1.2	+10.5	+ 3.4	+ 1.5	....	29.5
24–25	+15.3	+ 5.4	− 1.8	+ 3.6	+10.8	− 2.2	....	26.1
22–23	+10.2	+ 1.5	− 5.1	− 3.6	+19.0	− 6.3	....	29.2
20–21	+ 9.9	+ 3.5	+ 2.0	+ 5.5	+ 2.4	+ 2.5	....	15.9
18–19	− 0.7	− 9.3	− 2.7	−12.0	+ 6.6	− 3.3	13.3	5.9
16–17	−13.6	− 6.8	− 8.1	−14.9	+10.9	−10.0	30.4	....
14–15	−42.3	−15.9	−11.4	−27.3	+ 2.8	−14.1	72.3	....
37–38	− 7.1	+11.2	−22.0	−10.8	+29.8	−27.3	23.2	22.7
35–36	−12.9	− 3.5	−16.3	−19.8	+24.9	−20.2	36.6	12.0
33–34	−16.8	−15.8	−10.4	−26.2	+18.8	−12.9	45.5	2.0
31–32	−17.9	−19.2	− 4.2	−23.4	+10.0	− 5.2	42.3	....
29–30	−17.9	−17.0	+ 0.6	−16.4	+ 1.4	+ 0.7	34.9	....
27–28	−14.1	−11.6	+ 5.0	− 6.6	− 7.4	+ 6.2	25.7	....
25–26	− 9.4	− 5.3	+ 8.7	+ 3.4	−17.0	+10.8	26.4	1.4
23–24	− 4.7	+ 0.4	+11.0	+11.4	−23.3	+13.6	28.0	9.3
21–22	+ 3.0	+ 5.2	+13.1	+18.3	−29.5	+16.2	26.5	21.4
19–20	+ 0.8	+ 1.6	+ 3.1	+ 4.7	− 7.4	+ 3.8	6.6	6.2
17–18	+18.5	+ 9.2	+10.9	+20.1	−14.8	+13.5	....	41.2
15–16	+36.3	+16.8	+17.8	+34.6	−19.1	+22.1	....	75.2

* By proportion, 18 600 : 15 000.

Changes in Temperature do not seriously affect the stresses in the memb of a TRUE THREE-HINGED ARCH, or one with a tie-rod and rollers at one e as the change in geometrical shape is quite small. For the arch with a tie- and no rollers, the effect of changes in temperature may affect the support forces if the tie-rod is not so protected that it will change but little from average temperature. In most structures this is the case as the tie-rod is in under the floor of the building.

The Two-hinged Arch differs essentially in construction from the THR HINGED ARCH in having only two pins or hinges which are placed at the suppo Fig. 62 shows the form of truss which will be used in explaining the method finding the stresses in the members of the truss.

Supporting Forces. The SUPPORTING FORCES are inclined but can be solved into vertical and horizontal components. The vertical components readily found as they are the same as for a simple truss on two supports. horizontal components depend upon the AREAS of the members and their MOD OF ELASTICITY when the dimensions of the truss and the loading are known.

tal Thrust for Vertical Loads. This can be found from the formula

$$H_1 = \sum \frac{Sul}{AE} \div \sum \frac{u^2 l}{AE}$$

ymbols have the significance given on page 1086. But this contains n area A for each piece. For a preliminary trial the procedure is as the truss shown in Fig. 62, divide the span into twenty equal parts :enters of the divisions erect verticals. Through the points on these dway between the chords of the truss, draw a smooth curve as shown. ll be designated the AXIS OF THE ARCH. Number the points desig-

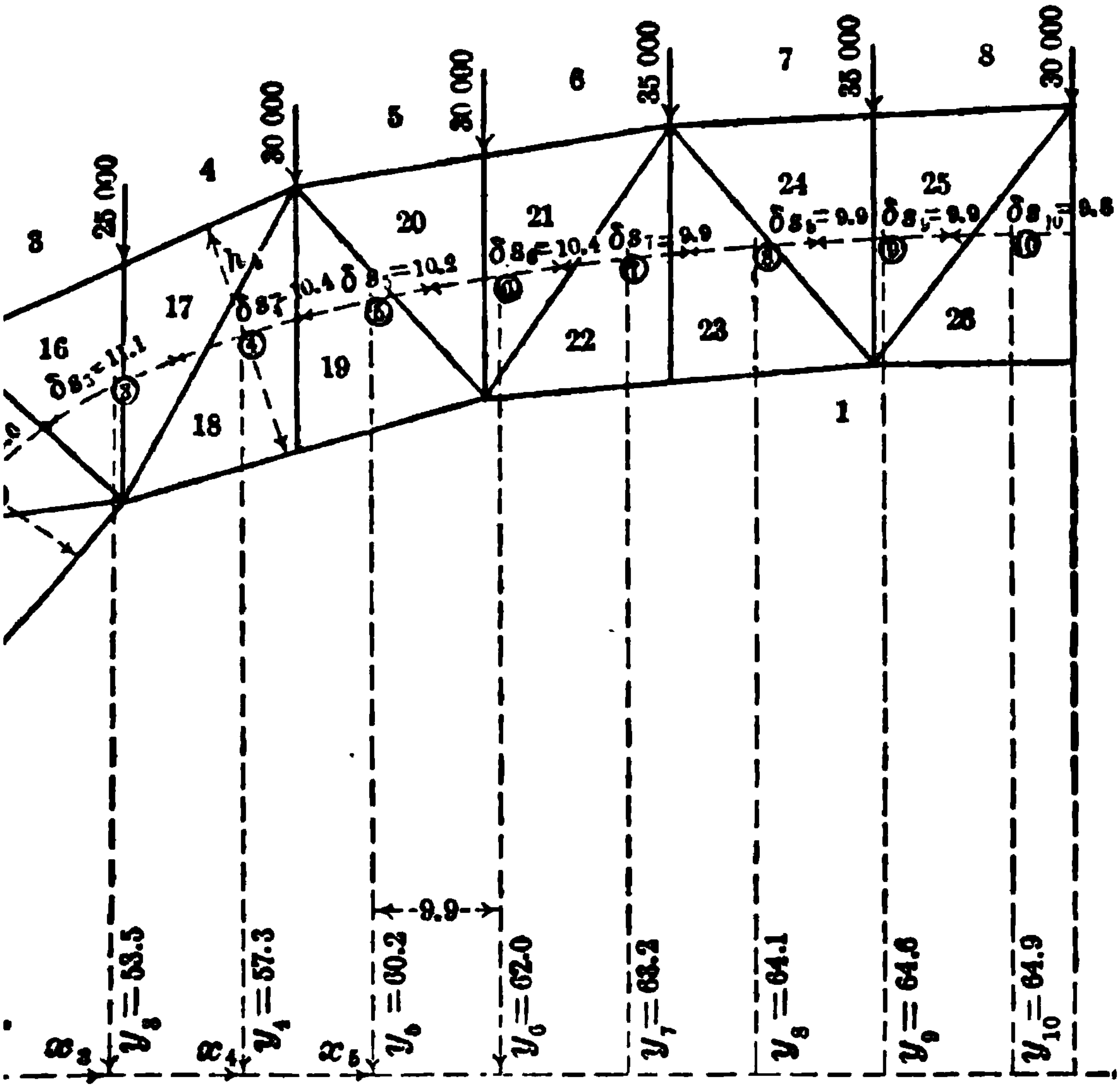


Fig. 62. Two-hinged Arch. Truss-diagram

:, 2, 3, etc., as shown in Fig. 62, and let x and y be their coördi- left support as the origin. Scale the length of the curve between the divisions so that y is practically the ordinate of the center of h of curve, and call this length of the curve δs. On a radial line iumbered 1, 2, 3, etc., scale the distances between the upper and calling the distance h and compute $\frac{1}{2} h^2 = I$, which expresses, the MOMENT OF INERTIA of the section when the chord-areas the web-members are neglected. Let M represent the BENDING point having the abscissa x, of the loads, considering the truss as on two supports; or, for a single load P, $M = Rx - P(x - a)$, than a, where a is the distance of the load P from the left sup- $\delta s \div EI$ is represented by ϕ the HORIZONTAL THRUST can be found ıla,

$$H_1 = \Sigma M y \phi \div \Sigma y^2 \phi$$

For the vertical loading shown in Fig. 62, the value of H_1 is 108 000 lb, and V_1 being one-half the total load, is 195 000 lb. The stresses in the members of the truss can now be found by the usual graphical method. The snow-load, if any must be treated in a like manner. The computations are considerably shorter since $\Sigma y^2\phi$ remains unchanged, regardless of the loading.

Wind-Loads. For WIND-LOADS the process is not changed very much. The value of M is the moment of the wind-loads, assuming the truss as HINGED at the right support and on ROLLERS at the left support. The value of V_1, which is vertical, is found by taking the sum of the moments of the wind-loads about the hinge at the right support and dividing this by the length of the span. The value of H_1 is found from the formula given above, and then the stresses are found by the ordinary stress-diagram. The MAXIMUM STRESSES are now found and the proper AREAS of the members determined.

The True Horizontal Thrust. The method just given is a close approximation to determine the AREAS of the pieces so that the correct formula for H_1 can be applied. This formula is

$$H_1 = \sum \frac{Sul}{AE} \div \sum \frac{u^2l}{AE}$$

where the symbols have the meaning already given. Applying this formula for the dead load shown in Fig. 62 and AREAS shown in Fig. 58, the value of H_1 is 110 600 lb, which is but little different from the value found by the approximate method.

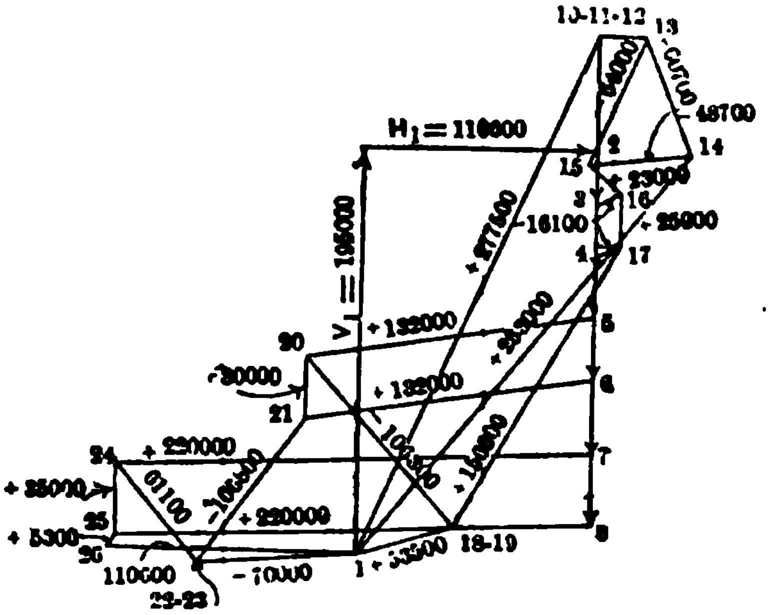


Fig. 62A. Two-hinged Arch. Stress-diagram

Dead-Load Stresses. The stress-diagram for the DEAD LOAD is shown in Fig. 62A. Considerable care must be exercised in drawing the stress diagrams, and their correctness should be checked by computing the stresses in one or more pieces. Compare Fig. 62A with Fig. 58A.

Changes in Temperature. Unlike the THREE-HINGED ARCH the TWO-HINGED ARCH is affected by CHANGES IN TEMPERATURE and the stresses which are produced by such changes must be provided for. $V_1 = O$ and H_1 is determined from the formula

$$H_1 = et^\circ L + \sum \frac{u^2l}{AE}$$

where e is the COEFFICIENT OF EXPANSION for the material composing the truss, t° the number of degrees CHANGE IN TEMPERATURE and L the SPAN of the truss. The other symbols have the significance already given. The above formula assumes that the truss-members are of the same kind of material. After H_1 has been found, the stresses can be determined by constructing the stress-diagram which will be of the shape shown in Fig 58B.

Tie-Rod. If a TIE-ROD connects the two supports of a TWO-HINGED ARCH the remarks made concerning such an arrangement for the THREE-HINGED ARCH apply here.

ted Arch has no hinges and is a type which is seldom employed by in the truss-form. The rigid analysis of a TRUSSED FIXED ARCH is und tedious, so a few formulas will be given, necessary for the solution WITH SOLID WEBS, such as PLATE-GIRDER ARCHES. These formulas plied to truss-forms, where the chords are approximately parallel, rious error. Midway between the top and bottom chords draw a ve, called the ARCH-AXIS, and designate the distance between its ends SPAN OF THE AXIS. Divide the span into n equal parts and at the these divisions draw perpendiculars until they cut the arch-axis.

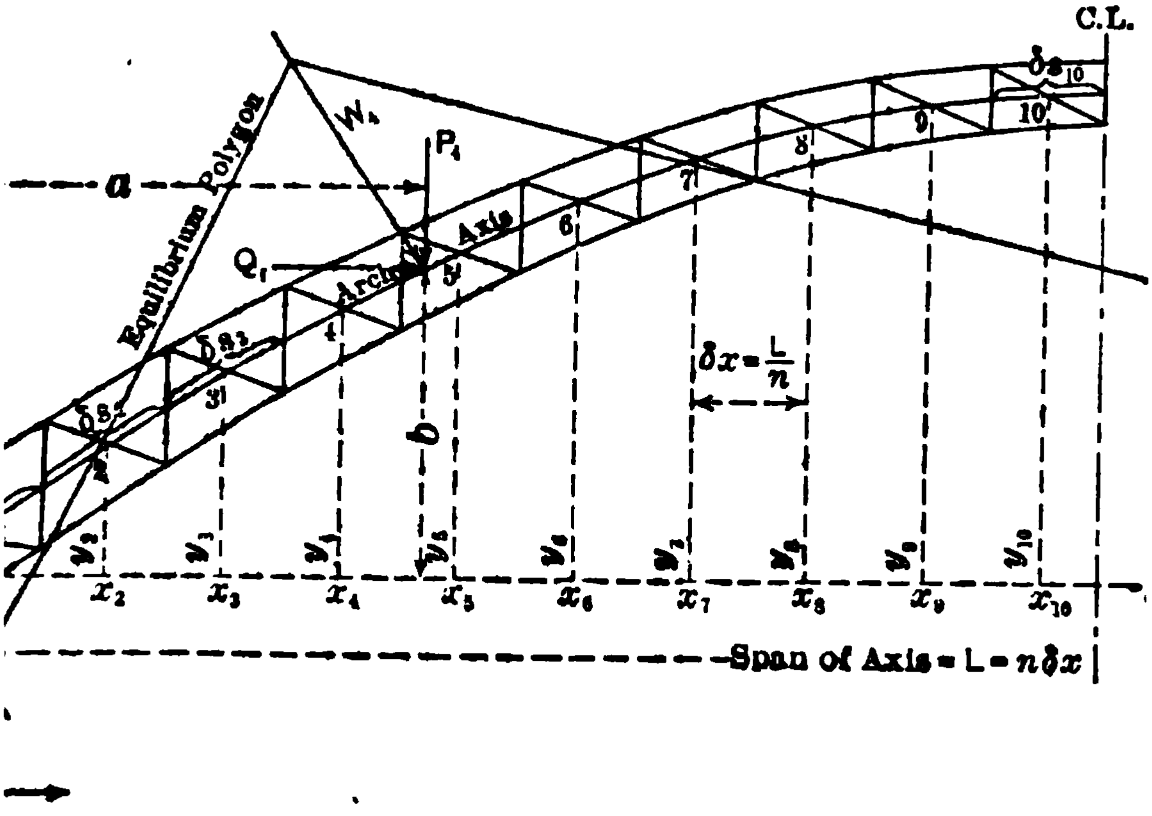


Fig. 63. Fixed Arch. Truss-diagram

points 1, 2, 3, etc., as shown by Fig. 63, which also indicates the e employed.

tion of H_1, V_1 and H_1y_1. The equilibrium-polygon for a single is shown in Fig. 63, in its true position with reference to the arch-ocates the point of application of H_1. The following formulas are pproximations for arches having a rise greater than one-eighth the

$$H_1 = \Sigma m_{xy} A'' \div \Sigma y A''$$

$$A'' = K\left\{y - \frac{\Sigma yK}{\Sigma K}\right\} \qquad K = \frac{\delta s}{EI}$$

$$\left.\begin{matrix} H_1 y_1 \\ H_2 y_2 \end{matrix}\right\} = H_1\frac{\Sigma yK}{\Sigma K} - \left\{\frac{\Sigma m_{xy}K}{\Sigma K} \pm \frac{\Sigma m_{xy}K(z-n)}{n^2\Sigma K - \Sigma z^2K}n\right\}$$

$$V_1 = \frac{H_1 y_2 - H_2 y_2}{L} + r_1 \qquad y_1 = \frac{H_1 y_1}{H_1}$$

l down from A when H_1y_1 is negative. Σ is the sum of quantities r each point on the arch-axis numbered 1, 2, 3, . . . n. For

$$\Sigma K = \left(\frac{\delta s}{EI}\right)_1 + \left(\frac{\delta s}{EI}\right)_2 + \left(\frac{\delta s}{EI}\right)_3 + \text{etc.}$$

I is the moment of inertia of the chords about an axis midway between th
The sections of the chords are to be taken on radial lines passing through point 1, 2, 3, etc. x and y are the coördinates the points 1, 2, 3, etc, in Fig. 63

$$x = z\frac{\delta x}{2} = z\frac{L}{2n}$$

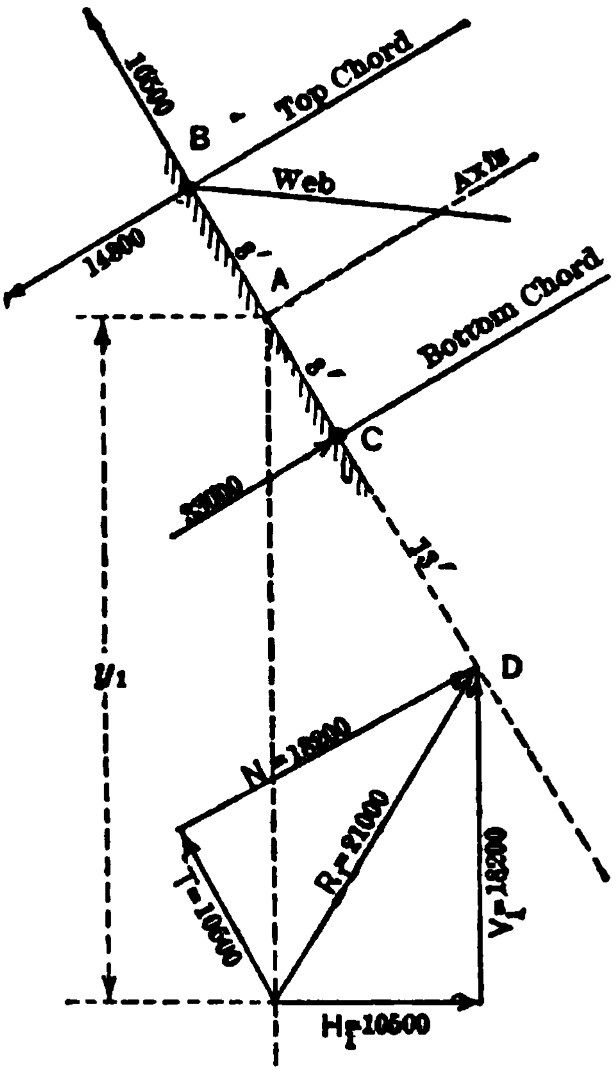


Fig. 64. Fixed Arch. Reactions

m_{xy} is the moment at the point on th arch-axis having the coördinates xy assum ing that the given loading is supported b the axis hinged at the right end and o rollers at the left end. r_1 is the reacti at the left support under the conditio specified for m_{xy}. In the above formul the only terms which depend upon th loading are those containing m_{xy} and r the others being constant for any give arch. While but one load has been use any number may be used by considerin m_{xy} and r_1 as the sum of the respecti quantities for each load.

Stresses. The STRESSES in the trus members can be found by the ordina graphical methods when H_1, V_1 and H are known. For example, assume th numerical values shown in Fig. 64. Th resultant of V_1 and H_1 is resolved int two components parallel and perpendi ular to the bottom-chord member a the support. Then T must act at th upper-chord joint as shown. The two reactions parallel to the bottom cho are found by moments. The stress-diagram can now be drawn beginning wi these forces and proceeding until the right support is reached.

Symmetrical Loading. When the loading is symmetrical, $H_1y_1 = H_2y_2$ an hence $V_1 = r_1$. Also

$$H_1y_1 = H_1\frac{\Sigma yK}{\Sigma K} - \frac{\Sigma m_{xy}K}{\Sigma K}$$

Changes of Temperature. For temperature-changes,

$$H_t = ct^\circ L + \Sigma yA''$$

$$H_1y_1 = H_2y_2 = H_t\frac{\Sigma yK}{\Sigma K}$$

$$y_1 = \frac{\Sigma yK}{\Sigma K} \qquad V_1 = 0$$

10. Arches with Solid Ribs

Arches with Solid Ribs. While this chapter considers TRUSSES only, may not be out of place to briefly consider ARCHES HAVING SOLID RIBS. T computations for V_1, H_1 and H_1y_1 remain unchanged, excepting that I now the moment of inertia of the radial section of the rib at points 1, 2, 3, etc.

esses. If x and y are the coördinates of any point on the gravity-rib, which should coincide with the arch-axis, the bending moment t is, for each load,

$$M_x = H_1 y_1 + V_1 x - H_1 y - P \overset{x>a}{(x-a)} - Q \overset{x>a}{(y-b)}$$

tive when y_1 is measured below A in Fig. 64.

$$S = \frac{M_x c}{I} + \frac{N_{xy}}{A}$$

he distance from the gravity-axis to the outermost fiber. For the REE-HINGED ARCHES, $H_1 y_1 = 0$.

ear. Let H_x be the algebraic sum of all the horizontal components of the section, V_x the algebraic sum of all the vertical components of the section and θ the angle which the radial section, upon which wanted, makes with the vertical. Then $T_x = V_x \cos\theta - H_x \sin\theta$.

ged Parabolic Arch. If the center line of the SOLID RIB is a PAR- $EI \cos\theta$ is a constant, the following simple formulas give the and H_1:

$$1 - k) - Q\frac{4f}{L}k(1-k)$$

$$\frac{L}{f} P[k(1-2k^2+k^3)] - Q\left\{1 - \frac{k}{2}[5(1-k-2k^2+4k^3) - 8k^4]\right\}$$

$$EI_0 et^\circ$$

$a + L$ (Fig. 63), f is the rise of the axis, P is the vertical load acting e horizontal load acting from left to right and I_0 is the moment of section of the rib at the crown.

abolic Arch. In like manner the following formulas apply for out hinges:

$$P(1-k)^2(1+2k) - \frac{12f}{L}Q(k-k^2)^2$$

$$\frac{15L}{4f}Pk^2(1-k)^2 - Q\left\{1 + k^2(-15 + 50k - 60k^2 + 24k^3)\right\}$$

$$\frac{L}{2}Pk(1-k)^2(5k-2) - fQ\left\{2k(1-k)^2(2 - 7k + 8k^2)\right\}$$

$$\frac{45}{4f}EI_0 et^\circ \qquad H_1 y_1 = \frac{15}{2f}EI_0 et^\circ$$

the factors containing k in the above formulas are given in tabular atise on Arches.*

ches, with solid ribs of constant cross-section and the center line rcle, may be considered by using formulas somewhat similar to r PARABOLIC ARCHES but very much longer and more complex. tables for their solution are given in the treatise on arches referred

on Arches, by Malverd A. Howe, John Wiley & Sons, Inc., New York.

11. Influence-Lines for Simple Beams and Trusses

An Influence-Line is a line showing the variation in any function at a section of a beam or for any member of a truss, caused by a single load movi across the span. For convenience the LOAD is usually considered as UNITY.

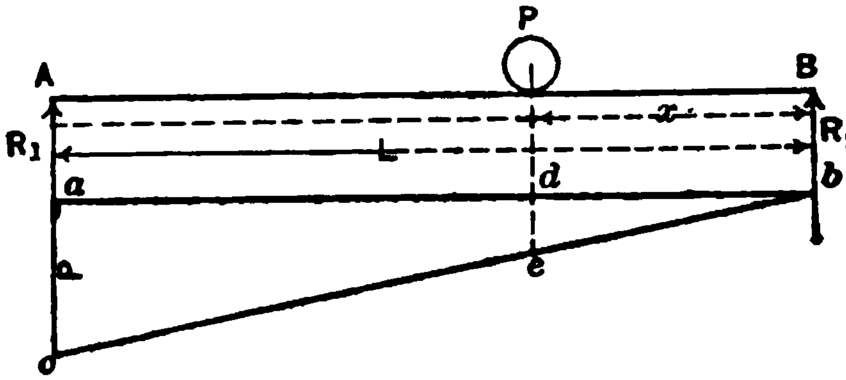


Fig. 65. Influence-lines. Reactions for Beams

Reaction for a Single Load. If the load P, Fig. 65, moves from A towa B, the left reaction, when P is distant x from B, is expressed algebraically $R_1 = Px \div L$, which is an equation of a straight line. If $x = 0$, $R_1 = 0$, i if $x = L$, $R_1 = P$. If we make $ac = P$ and draw the two straight lines ab and the ordinate de immediately below P is the value of R_1 for this position of If $ac =$ unity, then $R_1 = P\ (dc)$.

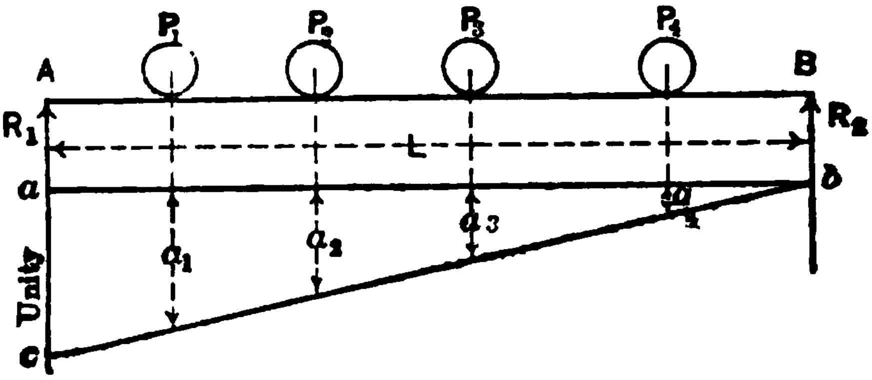


Fig. 66. Influence-lines. Reactions for Beams

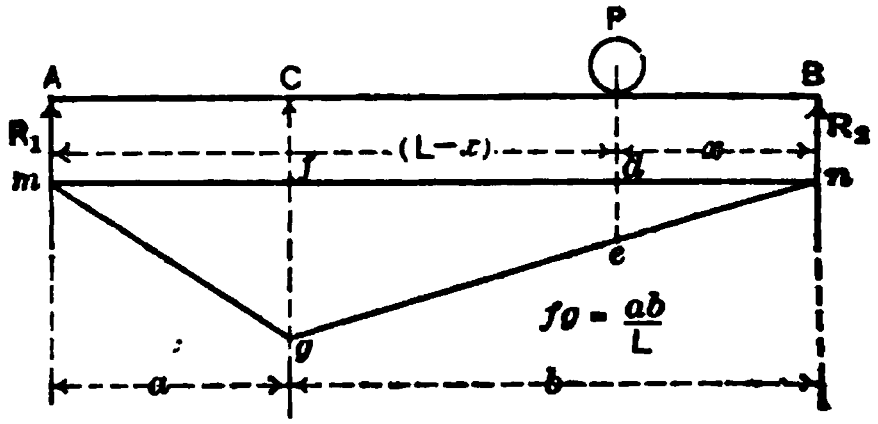


Fig. 67. Influence-lines. Moments for Beams

Reaction for More than One Load. The reaction for any numb concentrated loads can be found as shown in Fig. 66.

$$R_1 = P_1a_1 + P_2a_2 + P_3a_3 + P_4a_4$$

Bending Moment for a Single Load. The moment at C, Fig. 67, wh s on the right of C, is $M = R_1a = \dfrac{Pxa}{L}$. When the load is at C, $M = P$

at B, $M = 0$. For all positions of P upon the left of C, $M = R_2b$
$:)b$ —. When P is at A, $M = 0$, and when at C, $M = P\frac{(L-b)b}{L}$

. If in Fig. 67 the figure *mng* is drawn with $fg = \frac{(a)(b)}{L}$, then the
C for any load in any position is P (de).

Moment for Any Number of Concentrated Loads. The the point C for the loading shown in Fig. 68 is $M = P_1a_1 + P_2a_2$

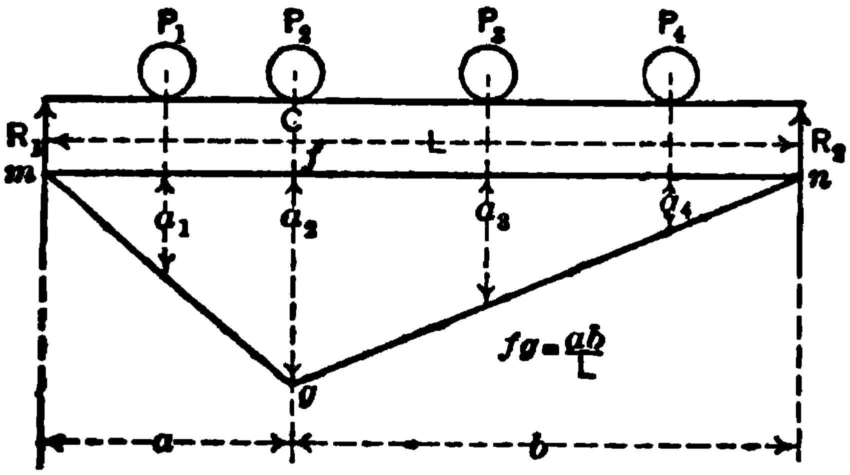


Fig. 68. Influence-lines. Moments for Beams

:4. This gives the moment at C for a given position of the loads, ot necessarily the GREATEST MOMENT which these loads may cause, r position may cause a greater moment. The greatest moment at when some concentration is at C. Let P be this concentration and

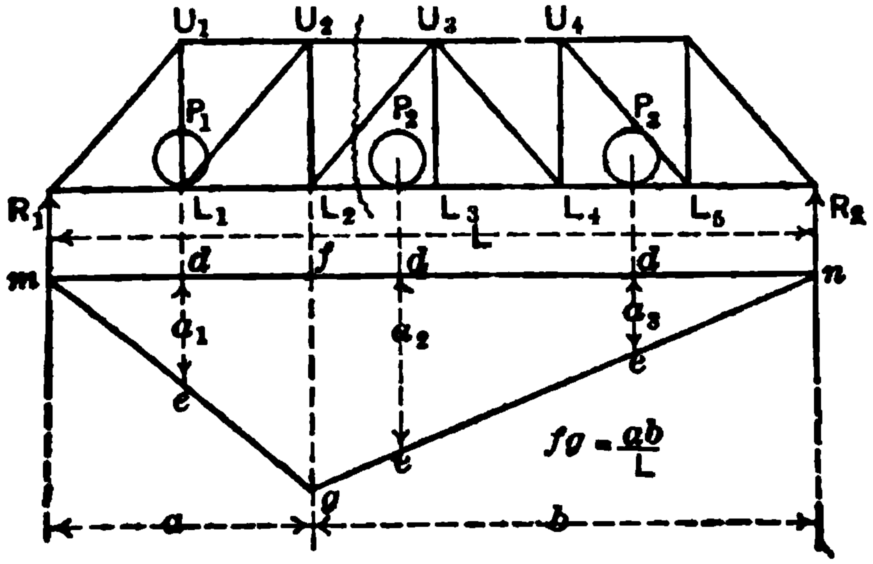


Fig. 69. Influence-lines. Moments for Trusses

›e divided into two parts, nP and mP so that $n + m = 1$, and n is ero and less than 1. The maximum moment at C will occur when

$$\frac{P_1 + P_2 + P_3 + P_4}{L} = \frac{P_1 + nP_2}{a}$$

n the beam where any given moving load causes the GREATEST ENT is so situated that the middle of the span is half-way between ter of gravity of the load. Since a concentration will always be a few trials will determine the proper concentration to use. For equal concentrated loads should be placed on the beam so that

the middle of the span is at the quarter-point between the concentrations. T MAXIMUM MOMENT falls under the concentration nearer the middle of the span

Chord-Member in Truss with One Set of Web-Members Vertical. Fig. 69 the top chord member U_2U_3 has its center of moments at L_2 and t bottom chord member L_1L_2 at U_2. The INFLUENCE-DIAGRAM for the mome at L_2 and U_2 is precisely the same as shown in Fig. 67. The moment produc by any load P is $P\ (de)$. As long as one set of web-members is vertical t INFLUENCE-DIAGRAM will be identical with that shown in Fig. 69, regardl of the inclination of the diagonals or the chord-members.

Chord-Members in Truss with Inclined Web-Members. The mome at points in the loaded chord, Fig. 70, have INFLUENCE-DIAGRAMS identical wi

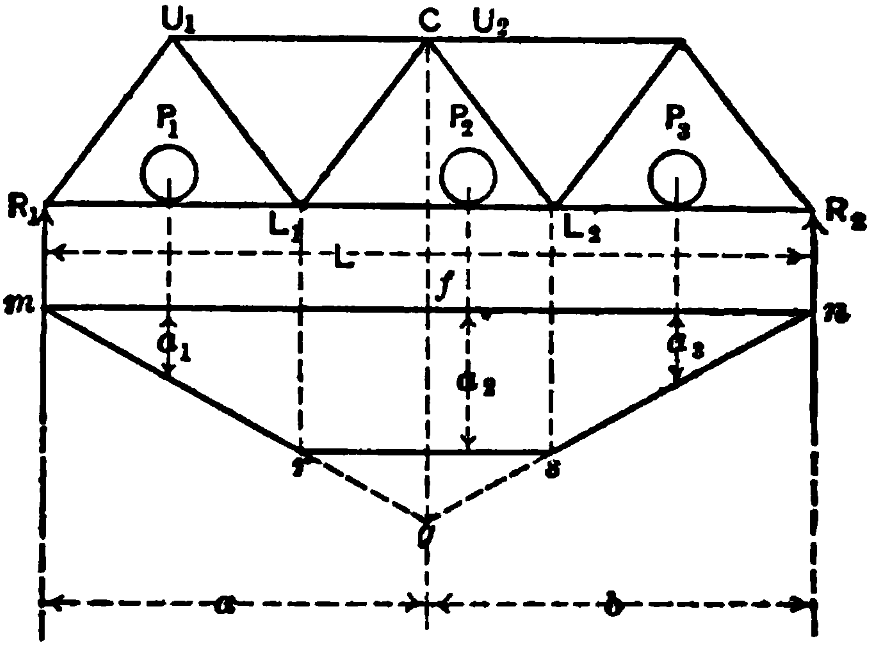


Fig. 70. Influence-lines. Moments for Trusses

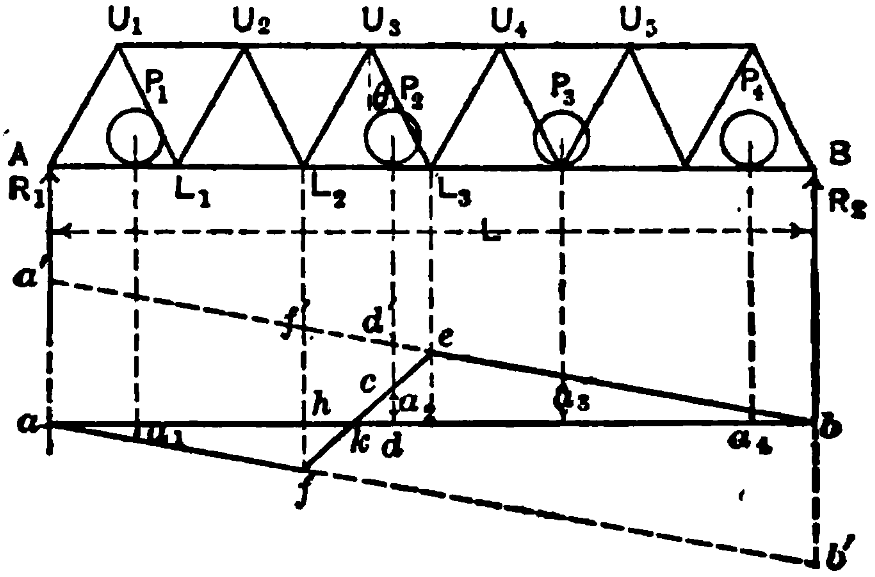


Fig. 71. Influence-lines. Shear for Trusses

that shown by Fig. 69. For the unloaded chord a slight modification must made. For example let U_2 be a center of moments, then if the loads were o beam, mgn would be the INFLUENCE-DIAGRAM (Fig. 70). For all loads on t left of L_1 and on the right of L_2 the diagram is correct and the moments at = P_1a_1 and P_3a_3. For loads between L_1 and L_2 draw the line rs. The mome at U_2 is P_2a_2.

Web-Members of Trusses with Parallel Chords. Fig. 71. The str in U_3L_3 equals the shear in the panel L_2L_3 multiplied by the secant of θ. T

DIAGRAM will be drawn for the shear. For any load between L_3 shear in this panel equals R_2; hence, with *ab* as a reference-line, *ba'* ENCE-LINE for R_2 and the shear is P_4a_4, P_3a_3, etc., until the point L_3 In like manner *af* is the INFLUENCE-LINE for R_1 and the shear for left of L_2 is P_1a_1. The shear for the loads P_2 between L_2 and L_3 amount of P_2 which is transferred to L_2. The INFLUENCE-DIAGRAM ions of P_2 on a span L_2L_3 is $ff'e$. The shear in this panel due to P_2 s P_2 $(d'c)$ or P_2a_2. A load at *k* produces no shear in the panel.

12. Secondary Stresses in Truss-Members *

y Stresses. In the determination of stresses in a truss it is med that they act along the GRAVITY-AXIS of each member; that axes of all members at any joint meet at a common point; that s are free to turn around this point, the joints being considered and that all loads, including the weights of the members, are he joints only. The stresses determined with these assumptions irect stresses, sometimes called PRIMARY STRESSES or MAIN STRESSES. tions made are not realized in practice and other stresses, called STRESSES, are induced. An eccentricity causing BENDING MOMENT e common case of the rivet-line not coinciding with the gravity-ngles connected with one leg only are used to resist direct stress. y-axes of members about a joint do not intersect at the same point nents are induced. The resistance of a joint to free angular move-truss deflects also induces bending moments. The weights of nd inclined members add slight BENDING-STRESSES to the direct ese members. At the supports there will be a RESISTANCE TO HORI-RMATION from temperature-changes and the deflection of the truss. of this resistance depends upon the coefficient of friction between the support, the vertical loads and the length of span. Members and imperfect workmanship are other causes of secondary stresses. the design and fabrication secondary stresses in ordinary roof-trusses ove causes need not be considered seriously. The main causes, secondary stresses are FAULTY DETAILS. The actual shearing-nes found in details is much more than the direct shearing-stress, CCENTRICITY IN THE LINES OF STRESS-ACTION. Eccentric riveted may not be wholly avoided but they should be reduced to a mini-history of bridge and building-failures is mostly a story of faulty tructure has seldom given way for lack of strength in the main ut if the strength of a structure is measured by the strength of art, it can be only as strong as the weakest detail. Because of ions that induce large secondary stresses, insufficient lacing of nembers and careless grouping of rivets, have all invited disaster. nce of the detailer's work is often underrated. What is usually e DESIGNING of a structure may be comparatively easy while the ay be difficult. A well-designed structure may be spoiled by . The detailer should be a designer, that is, a designer of details, ne time the designer should be thoroughly familiar with detailing.

* From Notes by Robins Fleming.

CHAPTER XXVIII

DESIGN AND CONSTRUCTION OF ROOF-TRUSSES

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1. Design of Wooden Trusses

Proportioning the Members. In Chapter XXVII it has been shown h the STRESSES in the members of a truss, supporting known LOADS, may be fou The next step is to PROPORTION THE MEMBERS for the stresses which they h to resist. The methods employed and the ALLOWABLE UNIT STRESSES are giv in detail in Chapters XI to XVI, inclusive. For example, tension-members considered on pages 385 to 400; steel strut-beams and tie-beams on pages 5 and 572; and wooden strut-beams and tie-beams on page 633. As a matt of convenience the UNIT STRESSES used in this chapter are given in the followi table in a condensed form. White pine is here used for the wooden trusses.

Table I. Allowable Unit Stresses Used in Truss-Design *

Material	Kind of stress	Safe unit stre lb per sq in
White pine.........	Tension with the grain....................	700
"	Tension across the grain..................	50
"	Compression on end-fibers................	1 100
"	Compression across the grain..............	200
"	Compression across the grain, round pins..	200
"	Columns† under 15 diam long............	1 100
"	Shear with the grain......................	100
"	Shear across the grain....................	500
"	Transverse, fiber-stress...................	700 ‡
Wrought iron.......	Rods in tension..........................	12 000
"	Bolts in shear............................	7 500
"	Bolts in bearing..........................	15 000
"	Bolts in bending, fiber-stress.............	15 000
Rolled steel........	Rods in tension..........................	16 000
"	Bolts in shear............................	10 000
"	Bolts in bearing..........................	20 000
"	Bolts in bending, fiber-stress.............	24 000
"	Beams in bending, fiber-stress.............	16 000
"	Beams in shear............................	10 000

* See also, the tables on pages 376, 412, 449, 454, 557, 647 and 1 200. These must modified, when necessary, to comply with building laws. White pine is used for t examples in this chapter because of the difficulties in making the joints owing to the r ative softness of the wood. If one can design a truss in white pine he will have no troub with the design of trusses constructed with other kinds of wood.

† See, also, Table I, page 449, and Table XVI, page 647.

‡ The Borough of Manhattan, New York, Building Code (1917), gives 1 200 for t value. Other values are about the same as in the table.

Inclined Surfaces of Wood. The normal intensity of the stress on inclin surfaces may be found from the empirical formula

$$r = q + (p - q)(\theta/90)^2$$

juals the permissible normal unit stress on this inclined surface, q the fibers, p that on the end of the fibers and θ the angle the inclined kes with the direction of the grain. For white pine this gives

$$r = 200 + \theta^2/9$$

Pins on End-Fibers.* For all practical purposes the permissible may be taken as the mean of p and q; or, for white pine

$$\tfrac{1}{2}(p+q) = 650 \text{ lb per sq in}$$

Columns over Fifteen Diameters Long. The formula † used in r and considered amply conservative by many engineers is the proved by the American Railway Engineering and Maintenance of ation in 1907. For white pine this formula is

$$S_1 = S\,(1 - l/60\,d) = 1100\,(1 - l/60\,d)$$

the permissible unit stress, S = the permissible compression on the = the length of the column in inches and d the least dimension ot ction of the column in inches.

lumns. For the shapes used in roof-trusses, the formula advocated wler in his specifications for roof-trusses is used in this chapter:

$$S_1 = 12\,500 - 500\,l/r$$

he permissible unit stress, l = the length of the column in feet, and radius of gyration of the cross-section of the column.

. The truss shown in Fig. 1, which is the queen truss shown in 53 and 54 in Chapter XXVII, is considered for this example. The n in the following table are used. The members RR are wrought-rods, not upset at the ends; and all other members are of white of the members in this truss is subject to transverse stress, so n and compression only, have to be considered:

. **Stresses and Dimensions for the Truss Shown in Figure 1**

er	Stress in pounds	Dimensions
....	+21 250	6 by 6-in white pine
....	+18 900	6 by 6-in white pine
....	+13 200	6 by 6-in white pine
....	+ 6 450	4 by 6-in white pine
....	+ 5 100	4 by 6-in white pine
....	−17 150	6 by 8-in white pine or Three 2 by 8-in pieces with ¾-in bolts, 2 ft on centers
....	−12 750	
....	− 9 100	One 1¼-in round rod

ls, **Fig. 1.** The tension in each rod is 9 100 lb. If the permis-2 000 lb, the section-area of each rod is 9 100 ÷ 12 000 = 0.76 sq in. f a 1⅛-in rod is 0.694 sq in; and of a 1¼-in rod, 0.893 sq in. The ld answer but the 1¼-in rod is preferred.

same unit stresses are used for flat and curved surfaces, Tables VII and to 431, of Chapter XII may be used.
rmulas and Tables based upon them, see Chapter XIV, pages 449 to 452.

Rafters, Fig. 1. The stress in the rafter at A is 21 250 lb and at B 18 900
but as it will be made of one piece, the size is governed by the greater str
The unsupported length is about 9 ft, and assuming the least dimension of
piece to be 6 in, $S_1 = 1\,100\left(1 - \frac{9 \times 12}{60 \times 6}\right) = 770$ lb per sq in. 21 250/77
27.6 sq in = the area of cross-section required, which is less than that of a 6 by
piece. A 6 by 6-in timber is actually 5½ by 5½-in, with a cross-sectional are
30.25 sq in, a little in excess of the area required. In general the NOMINAL
STANDARD sizes of timbers differ by about one-half an inch in each cross-dimens

Fig. 1. Queen Truss. (See, also, Figs. 4A, 10, 13 and 16 and Chapter XXVII,
3, 12, 53 and 54)

Member C, Fig. 1. The stress in this member is 13 200 lb and its unsuppo
length, 12 ft. In this case $l/d = 24$, when $d = 6$ in; $S_1 = 660$ lb per sq in.
required section-area is 13 200/660 = 20 sq in, and hence a 6 by 6-in timbe
used. The top-chord should have one dimension constant in order to facili
the making of good connections at the joints.

Braces, Fig. 1. The stress in the brace D is 6 450 lb and its unsuppor
length about 9 ft. A 4 by 6-in timber is first tried. Here $l/d = 27$ and $S_1 =$
lb per sq in. The required area, therefore, is 10.7 sq in and a 4 by 4-in tim
answers the purpose; but for additional stiffness and convenience in making
nections, a 4 by 6-in piece is used. Each brace, E, has a stress of 5 100 lb a
total length of 17 ft. If the braces are bolted where they cross the unsuppo
length may be taken as 8½ ft. It is evident that a 4 by 6-in piece is ample
each brace.

Bottom Chord, or Tie-Beam, Fig. 1. The maximum tension in the bott
chord is 17 150 lb in N. The permissible unit stress is 700 lb per sq in; he
the net section-area required is 17 150/700 = 24.5 sq in. A 2 by 12-in plan
continuous from end to end of the truss and without holes and notches, will t
care of the stress alone but will not permit of proper connections. A 6 by
piece is selected, but it may be necessary to substitute for it a 6 by 8-in p
when the connections are made and it is spliced in the middle. If the mem
is built up of planks, three 2 by 8-in pieces are required; and they must

bolted together by a pair of bolts every 2 ft of their length. If 24-lengths are used, the joints of the strands will be about 10 ft apart.

ı. For this example the truss illustrated in Fig. 2, which is the s shown in Figs. 4 and 24, Chapter XXVII, is considered. The s for dead load, wind and snow were found in Chapter XXVII and the following table. The rafters and the bottom chord support

. Scissors Truss. (See, also, Chapter XXVII, Figs. 4 and 24)

the joints and consequently must resist CROSS-BENDING stresses ECT STRESSES. The load on each piece is given in the table under ISVERSE.

Stresses and Dimensions for the Truss Shown in Figure 2

r	Stress, lb	Transverse load, lb	Dimensions, white pine
....	+8 000	1 000	Two 2 by 8-in planks
....	+6 600	1 320	Two 2 by 8-in planks
....	+1 890	....	One 2 by 10-in plank
....	+ 750	....	One 2 by 10-in plank
....	−4 350	....	Two 1 by 8-in planks
....	−2 530	....	Two 1 by 8-in planks
....	−1 875	470	One 2 by 10-in plank
....	−5 400	384	One 2 by 10-in plank
....	−1 875	....	One 2 by 10-in plank

g. 2. The piece *B* rather than the piece *A* is considered, as it is onger. The total vertical load on the piece acting as a beam is e horizontal span is about 8 ft. The bending moment at the center

is $\frac{1}{8}(1\,320 \times 8 \times 12) = 15\,840$ in-lb. If the depth of the piece is assumed be 8 in, the proper thickness is found from the equation, $15\,840 = \frac{1}{6} Sb$ $\frac{1}{6}(700 \times 8 \times 8 \times b)$, or $b = 2.12$ in. This neglects the component of the v tical load parallel to the rafter. Considering now the direct compression 6 600 lb and remembering that the sheathing is nailed to the rafter, the le dimension d of the piece is its depth, which may be taken the same as t used for the piece resisting the TRANSVERSE STRESS. The unsupported len of the piece is about 12 ft. Then $l/d = 18$, $S_1 = 770$ lb per sq in and the requi area of cross-section is $6\,600/770 = 8.6$ sq in. As the depth is 8 in, the thicknes about 1.1 in. Combining the two pieces, the total thickness is $2.12 + 1.1 = 3$ in, and a piece having the nominal size of 4 by 8 in is required. Since sheathing is nailed to the rafters, two 2 by 8-in planks may be used without creasing the stiffness of the member. While the above method of designin piece subject to TWO KINDS OF STRESS is not correct for all conditions wh occur in practice, the results are on the safe side, and the method has the vantage of being easily applied.

Member S, Fig. 2. Considering the transverse load first, the bending mom at the middle is found to be $\frac{1}{8}(470 \times 15.5 \times 12) = 10\,930$ in-lb. If the depth assumed to be 10 in, the required thickness, found from the equation 10 $= \frac{1}{6}(700 \times 10 \times 10 \times b)$, is 0.94 in; or, a board 1 by 10 in in cross-sectional a will carry the transverse load if prevented from twisting sidewise, which it ha tendency to do in this case where the ceiling is attached directly to the memb The side-stiffness will be further increased when the additional material resisting the tension is in place. The net area for the direct tension of 1 87 is 2.68 sq in, which requires a board 10 in wide and only a trifle over $\frac{1}{4}$ in th The total thickness becomes $0.94 + 0.27 = 1.21$ in, and it will therefore be n essary to use a 2 by 10-in plank.

Member E, Fig. 2. This is in compression, but the stress is quite sm being only 750 lb. The possible extension of the 2 by 10-in piece used for S next considered, to find if it can be extended and used here. The unsuppor length is about 6 ft, and the least dimension 2 in; hence $l/d = 36$, $S_1 = 44$ per sq in and the required area of the cross-section becomes less than 2 sq The 2 by 10-in piece is therefore ample.

Members T and T_1, Fig. 2. Inspection shows that a 2 by 10-in plan quite sufficient for these pieces.

Member D, Fig. 2. The unsupported length is about 7 ft. Then, for d in, $l/d = 42$, $S_1 = 330$ lb per sq in and the required section-area is $1\,890/33$ 5.73 sq in. A piece 2 by 10 in is more than sufficient; but as this size allow a simple prolongation of the pieces T and T_1, a piece of this dimension is us

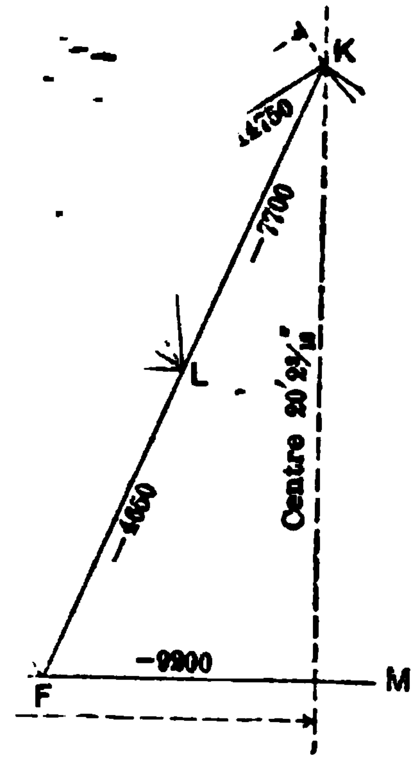
Members F and H, Fig. 2. For the piece F a net area of $4\,350/700 =$ sq in is required; or a board 1 by 8 in in cross-section may be used. For c venience in construction, two 1 by 8-in boards are chosen. It is evident that same arrangement can be made for the piece H.

Caution. Since this truss is a SCISSORS TRUSS, the horizontal deflection the supports should be determined, and, if this is an appreciable quantity, members as designed above, should be increased in size or their lengths chan in framing; this is so that the span, after the truss is loaded and the deflec has taken place, becomes the distance between the supports. This is discus in Chapter XXVII, pages 1085 to 1087.

Example 3. For this example the HOWE TRUSS shown in Fig. 3 is conside The vertical load is assumed to be $46\frac{1}{2}$ lb per sq ft on the top-chord and per sq ft on the bottom-chord. For trusses spaced about 15 ft 8 in on cent

... is 2½ by 2 by ¼ in; and while ⅝-in ... rivets, which are the largest that ... wooden trusses, it is economical ... the same straight line, but

...ions of a truss ...ght and con-



... with Stresses

...ny of the members. For conven-
...mbers and final sections are arranged

... or the Half-Truss Shown in Figure 5

	Net area required, sq in	Make-up of member
	1.06	Two 2½ by 2 by ¼-in angles Net area = 1.70 sq in
..	....	
..	....	
...	....	
....	....	
.....	....	
72	....	Two 2½ by 2½ by ¼-in angles, and one 10 by ¼-in plate
144	....	Two 2½ by 2 by ¼-in angles
72	....	

This member has the maximum stress of the bottom- ...e used up to the joint F and possibly for the entire length ...t area required is 16 900/16 000 = 1.06 sq in, or the net ...ngle is 0.53 sq in. One leg of the angle is riveted to the ...in rivets which is assumed to cut out a section ... the

chord 10 by 12 in. A single piece of this size, nearly 50 ft long, is difficul obtain; so at least one splice is necessary. If planks are substituted it requ six 2 by 12-in pieces to give an equivalent area.

Inclined End-Post, Fig. 8. The stress in this post is 33 450 lb and its supported length about 9.75 ft. An 8 by 10-in piece is tried first, the dimension being the same as one dimension of the chords. Then $l/d = 14$ $S_1 = 825$ lb per sq in and the required area of cross-section becomes 33 450/82 40.54 sq in, which is about one-half the cross-sectional area of an 8 by 10-in pi If $d = 6$ in, there results $l/d = 19.50$, $S_1 = 740$ lb per sq in and the required = 45.2 sq in, which is a much smaller cross-sectional area than that of a 6 10-in timber.

Intermediate Diagonals, Fig. 8. For the first diagonal a 6 by 6-in piec tried. The required section-area is 19 630/740 = 26.5 sq in, which is well wi the section-area of a 6 by 6-in timber. A 4 by 4-in timber could be used for next brace, but it is better to use either a 6 by 6-in timber or one 4 by 6 i

Purlins, Figs. 1 and 4A. While the stresses given for the members of truss shown in Fig. 1 are based upon a vertical loading covering the effec dead load, snow-load and wind-load, the w load is separated from the others in orde illustrate the method to be followed in de ing a purlin when the plane of the load is parallel to one of its sides. The trusses of type illustrated in Fig. 1, are 15 ft on cen This distance, therefore, is the span of purlin. The purlin at joint 2, Fig. 1, h vertical loading of 4 500 lb and a wind-l acting normal to the roof, of 2 000 lb. inclined loading, resolved parallel to b a Fig 4A, and combined with the vertical l gives for the total, parallel to d, 4 500 + 1 = 5 900 lb; and for that parallel to b, 1 lb. If then, loads are assumed to act thr the center of gravity of the purlin-section,

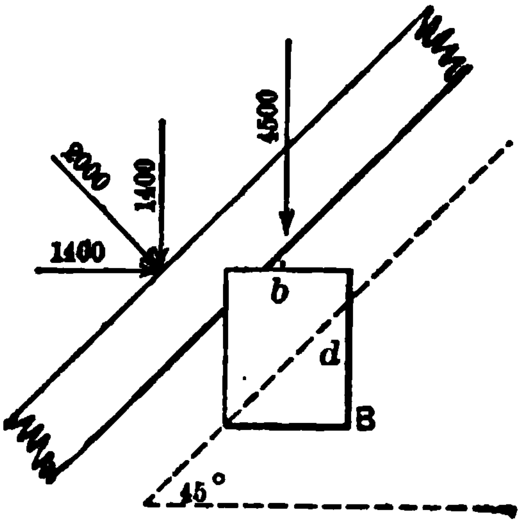


Fig. 4A. Purlin-design for Joint 2 of Truss Shown in Fig. 1

produce both tension in the fiber at B and compression diagonally opposit The bending moment at the middle of the purlin due to the vertical load (5 900 × 15 × 12) = 132 750 in-lb; and that due to the horizontal load (1 400 × 15 × 12) = 31 500 in-lb. It is assumed that $b = 8$ in and $d = 1$ Then the fiber-stress at B, due to the first moment is

$$S' = 6 \times 132\,750/bd^2 = 996 \text{ lb per sq in}$$

The fiber-stress at B, produced by the second moment is

$$S'' = 6 \times 31\,500/b^2d = 295 \text{ lb per sq in}$$

The total fiber-stress is 996 + 295 = 1 291 lb per sq in. This is 91 lb in e of the permissible fiber-stress in the most conservative practice and in som building laws for white oak. If a 10 by 10-in timber is used the fiber- is 986 lb per sq. in, and if the piece is 10 by 12-in, the fiber-stress become lb per sq in.

2. Design of Steel Trusses

General Considerations. The members of the ordinary STEEL TRUSSES are composed of two rolled ANGLES placed back to back and a joints each piece is connected to GUSSET-PLATES by RIVETS. The size o

mallest angle permissible in good practice is 2½ by 2 by ¼ in; and while ⅝-in vets are often used, it is better to use ¾-in rivets, which are the largest that in be used in a 2½-in leg of an angle. As in wooden trusses, it is economical) use the same sizes for all members which are in the same straight line, but is is not always done.

Example 4. Fig. 5 shows a FAN TRUSS of the form and dimensions of a truss ed for supporting the roof of a machine-shop. The loading is light and con-

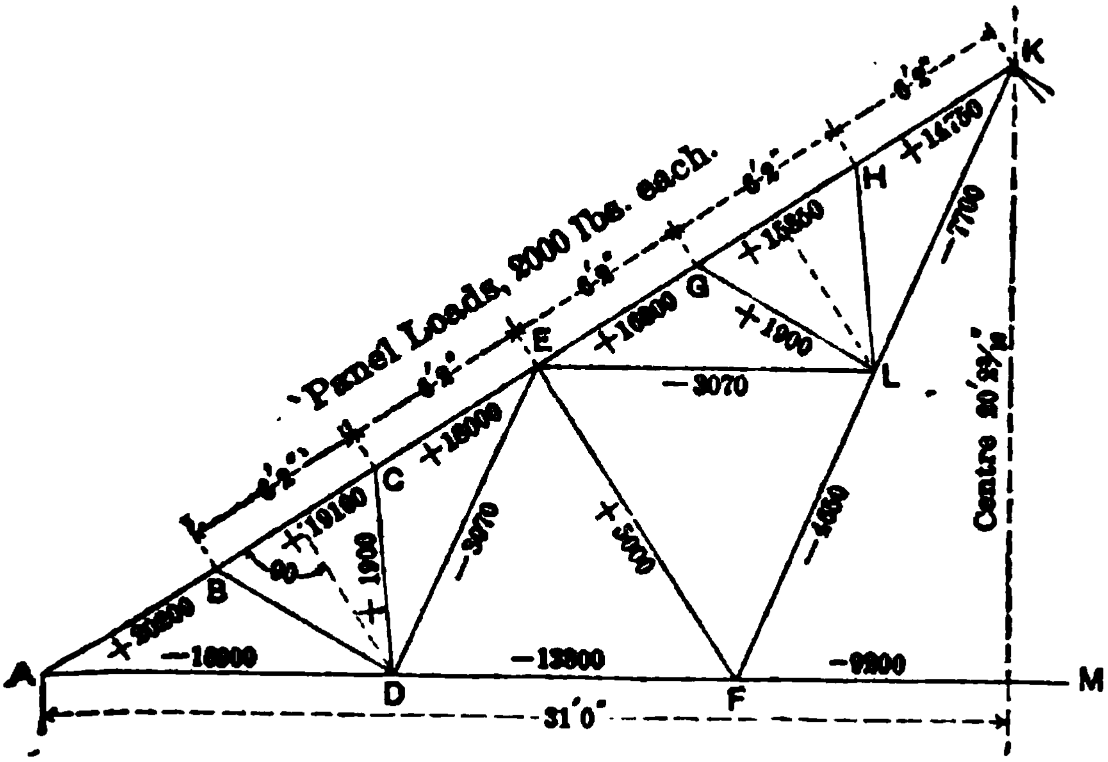


Fig. 5. Fan-truss Diagram with Stresses

quently the stresses are quite small in many of the members. For convenie ce the stresses, lengths of compression-members and final sections are arranged tabular form.

able IV. Stresses and Dimensions for the Half-Truss Shown in Figure 5

Member	Stress, lb	Approximate length, in	Net area required, sq in	Make-up of member
D..........	−16 900	....	1.06	Two 2½ by 2 by ¼-in angles Net area = 1.70 sq in
F..........	−13 800	....	....	
M	− 9 200	....	....	
L.	− 4 650	....	....	
K..........	− 7 700	....	....	
E and EL...	− 3 070	....	....	
B..........	+20 200	72	....	Two 2½ by 2½ by ¼-in angles, and one 10 by ¼-in plate
F..........	+ 5 000	144	....	Two 2½ by 2 by ¼-in angles
D..........	+ 1 900	72	....	

Member AD, Fig. 5. This member has the maximum stress of the bottom-rd and its size will be used up to the joint F and possibly for the entire length he chord. The net area required is 16 900/16 000 = 1.06 sq in, or the net ion-area of one angle is 0.53 sq in. One leg of the angle is riveted to the et-plate with ¾-in rivets which is assumed to cut out a section ⅞ in by the

thickness of the angle. From Table XI, page 365, we find that a $2\frac{1}{2}$ by 2
$\frac{1}{4}$-in angle has an area of 1.06 sq in. The area to be deducted on account
one rivet-hole is $\frac{7}{8}\times\frac{1}{4} = \frac{7}{32} = 0.22$ sq in. This leaves for the net area of t
angle $1.06 - 0.22 = 0.84$ sq in, which is well above the required area. As th
is the smallest angle which can be used and as all the other tension-membe
have less stress than AD, the tension-members will be made uniform throug
out. With the exception of FK, many designers would use but one angle f
the web-members. While the net area is ample for the stresses, yet it is po
practice, as one angle produces a ONE-SIDED PULL on the gusset-plates.

Member AB, Fig. 5. This piece has the maximum stress of the top-chor
a compression of 20 200 lb and a transverse load of 2 000 lb. The COMBINI
EFFECT OF THE TWO LOADINGS in this case must be determined in a mann
quite different from that followed for wooden construction. The maximu
fiber-stress must not exceed that found from some column-formula as, f
example, $S_1 = 12\,500 - 500\ l/r$. The maximum fibe
stress S may be found, approximately, from the expre
sion $S = P/A + Mc/I$. In this equation, P is t
direct compression, which is 20 200 lb in this cas
A the area of the section of the piece, I the mome
of inertia of this section, c the distance of the oute
most fiber of the section which is in compression fro
its gravity-axis and M the maximum bending mome
produced by the tranverse load. The PRINCIPAL AX
of the section must lie in the plane of the transver
loading, if the above formula is used. For sy
metrical sections, as two angles back to back, an
beam, or a channel, the principal axes are AXES
SYMMETRY, and the values of I and c are readi
found from the properties of rolled shapes tabula
in Chapter X. The first trial-section is that sho
in Fig. 6, consisting of two $2\frac{1}{2}$ by $2\frac{1}{2}$ by $\frac{1}{4}$-in angl
and one 10 by $\frac{1}{4}$-in plate. To find the moment
inertia, I, and the radius of gyration, r, the center of gravity of the section
found first. The distance X of the center of gravity from AB (Fig. 6) is fou
from Equation (2), page 295,

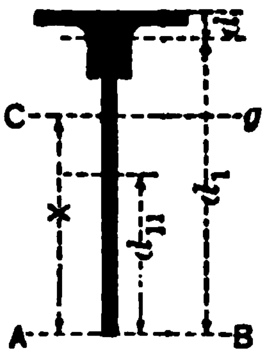


Fig. 6. Section through Rafter-member of Truss Shown in Fig. 5. (Axis at d_1 from AB, through c. g. of angles; axis at Cg, through c. g. of sections; axis at d_{11} from AB, through c. g. of plate.)

$$X = \frac{\text{area of plate} \times d_{11} + \text{area of angles} \times d_1}{\text{area of entire section}}$$

From the properties of angles, Table XII, page 367, the distance from the ba
to the center of gravity of the angle is $d = 0.72$ in. $I = 0.7$ and the area of
two angles $= 2.38$ sq in. The plate does not usually extend to the back of
angles, a clearance of from $\frac{1}{8}$ to $\frac{1}{4}$ in being allowed. A clearance of $\frac{1}{4}$ in
assumed.

Then, $\quad d_1 = 10.25 - 0.72 = 9.53$ in, and $d_{11} = 5$ in

Hence, $\quad X = \dfrac{2.5 \times 5 + 2.38 \times 9.53}{2.50 + 2.38} = 7.21$ in

The value of the moment of inertia I, about Cg as an axis, is found as foll
(Chapter X):

For the plate (page 335), $\quad \dfrac{th^3}{12} = \dfrac{0.25 \times 10^3}{12} = 20.80$

Eq. (3) (page 338), $\quad A\,(X - d_{11})^2 = 2.5\,(2.21)^2 = 12.21$

) angles (page 367) $2 \times 0.70 = 1.40$
ge 338), $A(d_1 - X)^2 = 2.38\,(2.32)^2 = 12.81$
ire section, $I = 47.22$

section, $I = Ar^2$ or $r = \sqrt{I/A}$, hence for this section $r = \sqrt{47.22/4.88}$ (See Equation (2), page 333.) The distance to the outermost fiber ion from the axis Cg is 3.04 in $= c$. There is now sufficient data to e actual fiber-stresses due to the loading and also the permissible bending moment produced by the transverse load is

$$M = \tfrac{1}{8}(2\,000 \times 6.16 \times 12) = 18\,480 \text{ in lb}$$
$$S = 20\,200/4.88 + (18\,480 \times 3.04)/47.22 = 5\,330 \text{ lb per sq in}$$
$$S_1 = 12\,500 - 500 \times 6.16/3.11 = 11\,510 \text{ lb per sq in}$$

s that the actual fiber-stress is very much smaller than the allowss, but as we have used minimum-size angles and a minimum thickplate, the only way to reduce this section is to use a smaller plate. asible because of the requirements for making proper connections
The above analysis assumes that the member is prevented from vise by the roof-covering. If such is not the case, r will have to d for a vertical axis through the center of gravity of the section. the moment of inertia,

$$bt^3/12 = (10 \times 0.25^3)/12 = 0.013$$
s, $$2 \times 0.70 = 1.400$$
$$2.38\,(0.72 + 0.125)^2 = 1.699$$
section, $I = 3.112$

$$r = \sqrt{3.112/4.88} = 0.6377 \text{ in}$$
$$S_1 = 12\,500 - 500 \times 6.16/0.6377 = 7\,670 \text{ lb per sq in}$$

less than the value of S_1, and hence this sections fulfills all the considering the unsupported length verticallv and sidewise as

, Fig. 5. Taking two $2\tfrac{1}{2}$ by 2 by $\tfrac{1}{4}$-in angles with the $2\tfrac{1}{2}$-in ack, the least value of $r = 0.78$ in (Table XVI, page 371).

$$S_1 = 12\,500 - 500 \times 12/0.78 = 4\,810 \text{ lb per sq in}$$
$$5\,000 \div 4\,810 = 1.04 \text{ sq in, required.}$$

he two angles used is $2 \times 1.06 = 2.12$ sq in (Table XVI, page 371).

Fig. 5. The stress in this member is very small and one angle lfill the requirements. For one angle, $2\tfrac{1}{2}$ by 2 by $\tfrac{1}{4}$ in, the least ole XI, page 365) and $S_1 = 5\,300$ lb per sq in, indicating that this rge excess of strength. As pointed out above, it is better to use

Ratio. The best specifications limit the ratio of the least dimenpported length of a compression-member to 50, unless the allowas given by the column-formula is decreased. The member EF nd about 144 in long, so that its length is 57.6 times its least there is a great excess of area, the actual unit stress is much by the formula.

The compression-members made up of two angles and designed the preceding paragraphs, have been considered as if acting as is clear that the various parts must be so fastened together that ce will buckle. If l is the unsupported length of the member

as a whole, r the corresponding least radius of gyration, r' the least radi
gyration for any part and l' the unsupported length of the part, or the dis
between stay-rivets, there is the following relation:

$$l'/r' = l/r \text{ or } l' = lr'/r$$

For the member *EF* $l' = 144 \times 0.42/0.78 = 78$ in.
Practice reduces this to 2 or 3 ft.

Tension-Members, also, should be riveted together in a similar mann
make the parts pull together.

Example 5. The next truss considered is the FINK TRUSS shown in F
in which two angles are used for all members and ¾-in rivets at the co
tions.

Member *AC*, Fig. 7. For a unit stress of 16 000 lb per sq in, the net
required is 21 800/16 000 = 1.36 sq in. Two 2½ by 2½ by ¼-in angles h
section-area of 2.38 sq in (Table XII, page 367). Deducting 2 (⅞ × ¼) =
sq in, the net section becomes 1.94 sq in, while the required area is 1.36

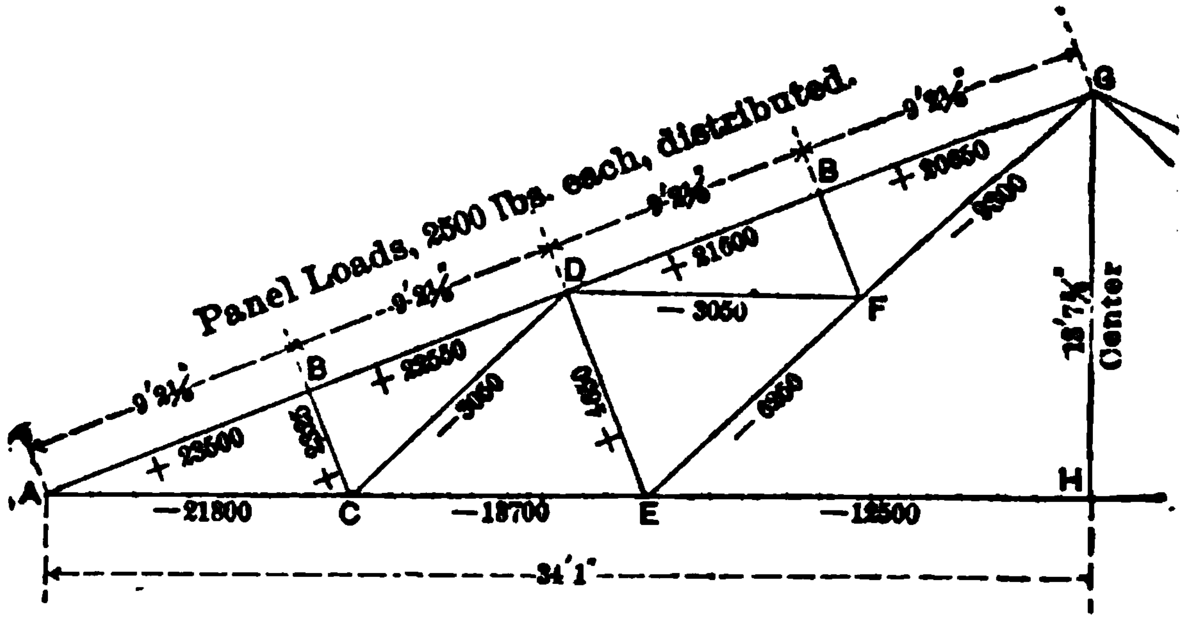


Fig. 7. Fink Truss. (See, also, Figs. 22, 22A, 22B, 22C, and 22D)

Members *CE* and *EH*, Fig. 7, are composed of angles of the same size.

Members *CD*, *DF*, *EF* and *FG*, Fig. 7, are made of two 2½ by 2 by
angles with a net area of 2.12 sq in. This greatly exceeds the required a

Members *BC*, *BF* and *DE*, Fig. 7. From the preceding example it is ev
that a pair of minimum-size angles will be quite sufficient. Two 2½ by
¼-in angles, having a section-area of 2.12 sq in, are used.

Member *AB*, Fig. 7. For this member in which there is a direct com
sion of 23 500 lb and a transverse stress due to a load of 2 500 lb, a prelim
trial is made with two 5 by 3½ by ⅜-in angles, with the 5-in legs back to bac
separated by a ¼-in gusset-plate. The moment of inertia about an axis p
through the center of gravity of the two angles and parallel to the short
is (Table XI, page 363) 7.78* and the corresponding radius of gyration
in. About a vertical axis the radius of gyration is (Table XVI, page
1.42 in, which is the least radius to be used in the column-formula.

* It will be noticed that some values given for the properties and the sizes of angl
in this example differ slightly from those given in the tables referred to, as the sectio
I, r, x, etc. This changes the result very slightly and is due to variations in the d
figures of values in different editions of manufacturers' handbooks. Editor-in-c

oduced by the 2 500-lb load at the center of the member is ⅛ × 12) = 34 500 in-lb. The section-area of the two angles is (page n.

$= [23\,500/6.10] + [(34\,500 \times 1.61)/7.78] = 10\,990$ lb per sq in
$= 12\,500 - (500 \times 9.2)/1.42 = 9\,260$ lb per sq in

ss than S, it is seen that the angles selected are a little too light. sing angles of greater thickness it will be better to select a larger 6 by 3½ by ⅜-in angles are used (page 363)

$= [23\,500/6.86] + [34\,500 \times 2.04)/12.86] = 8\,890$ lb per sq in
$= 12\,500 - [(500 \times 9.2)/1.34] = 9.070$ lb per sq in

hat there is ample strength and stiffness and that the area is in-76 sq in. If two 5 by 3½ by 7/16-in angles had been used, the area een increased 0.96 sq in (page 363). The least radius of gyration pression for S_1 assumes that the angles will be separated by ¼-in If thicker gusset-plates are used, the value of r will increase.

tails. The use of UNIFORM SIZES for members in the same straight ical and adds rigidity to the truss. The angles can be furnished of 60 ft and over, thereby reducing the labor of cutting them and number of rivets and the size of the gusset-plates. The portion EG shown by Fig. 7 would be completely riveted up in the shops, hree joints to be riveted at the building. In general, any truss outside dimension not exceeding 10 ft, can be shipped by rail. he location of the splices.

3. Joints of Wooden Trusses

of any truss should be proportioned with as much care as is used the sizes of the members, so that the truss will be equally strong s. The general principles and methods for designing joints are hapter XII and illustrated by examples. To further explain the ethods of design of some of the joints for the trusses shown in are added in this chapter.

1. This is the most important joint in the truss. There are this joint, but only a few of them are illustrated. Fig. 8 shows a JOINT. The rafter rests in a notch in the bottom chord and is y one or more rolled-steel bolts. These bolts are perpendicular he rafter, and the stresses in them are found graphically by the ig. 8) in which ac is perpendicular to the SCARF-CUT or SEAT of e tension in the bolts is found to be 31 550 lb, and with a permis-16 000 lb per sq in, the net section-area required is 1.97 sq in, nds to one 1⅞-in bolt (Table II, page 388). The WASHER, bear-grain of the rafter, will have an area of 31 550/200 = 158 sq in ince the top-chord is actually but 5½ in wide, the length of the 8 in. Such a plate would look out of proportion with one bolt, ts are substituted, having a net section-area of 2.10 sq in (Table Two bolts are placed near each end of the plate and one bolt middle. The bolts are spaced about 9½ in apart. The thick-e may be taken as one-fifth the distance from the end of the s of the first pair of bolts. This distance is about 3.4 in; hence 0.67 in. A ¾-in plate is used. The lower end of each pair of l with a PLATE-WASHER bearing upon the inclined surface of the ER as shown. The ANGLE OF INCLINATION approximates 45°

and hence the allowable pressure on the wood is 500 + (1 400 − 500) ¼ = 72 per sq in. (See Table VI, page 454, Table XVI, page 647, and the equatio page 1138.) The pair of bolts carry a tension of 31 550 × ¾ = 12 620 lb, this stress requires a plate having an area of 12 620/725 = 17.4 sq in, w will be provided by a plate 5¼ by 4 by ¾ in. For the single bolt, a 4 by CAST-IRON BEVELED WASHER is used, having a ¾ in lug let into the bol to take the horizontal component of the pull in the bolt. To prevent the bol slipping on the bottom chord, two OAK KEYS are employed. (See Table page 454, and Table I, page 1138, for permissible unit stresses.) The horizo component of the pull in the bolts is about 22 300 lb, and for one key, 11

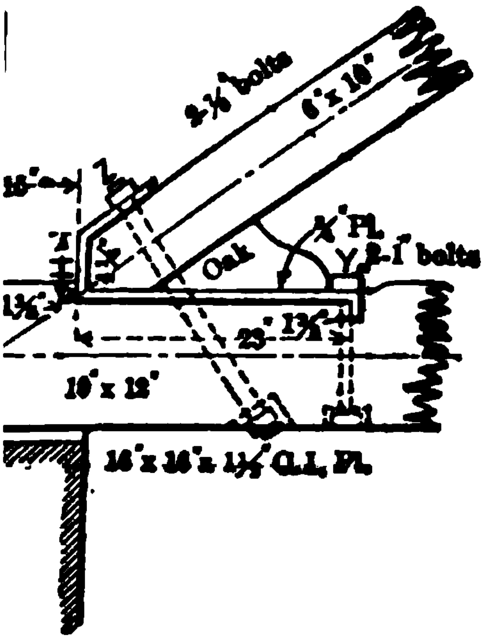
Fig. 8. Detail of Joint 1, Fig. 1

lb, and taking the actual thickness of 6 in to be 5½ in, each inch in lengt the key will safely carry 5½ × 200 = 1 100 lb in longitudinal shear (Tab page 412). The keys will, therefore, be 10 in long. The ends of the keys against the notch in the white-pine chord, and for end-bearing each in depth of the notch carries 5½ × 1 100 = 6 050 lb. The depth of the n therefore, is 1.8 in in the chord. In the bolster each inch in depth of n carries 5½ × 1 400 = 7 700 lb (Table XVI, page 647), and the depth of n is 1.43 in. This makes the total thickness of the keys 1.8 + 1.43 = 3.23 in, say 3½ in. The size of the keys is 3½ by 5½ by 10 in. The SPACING OF KEYS is governed by the longitudinal shear of the white-pine chord. E inch in length carries 5½ × 100 = 550 lb (Table I, page 1138), and the distance between keys is 11 150/550 = 20 in. The various dimensions above will probaby appear large to many. The large dimensions are du the timber used. If long-leaf yellow pine had been employed, many of dimensions would have been materially smaller. The ANGLE-BLOCK d shown in Fig. 9a, makes a much better connection in this case. 1½ becomes 1¼ in and 15 in becomes 13 in. The net area of the bottom c should now be determined to see if it is sufficient to take the tension.

Wall-Plate. As a rule it is a good idea to place the WALL-PLATE, w receives the common rafters, just above the bottom chord as shown. affords an opportunity to get at the nuts on the bolts to tighten them a wood shrinks. The BEARING OF THE TRUSS on the brickwork should be

ıd a STONE OR METAL PLATE provided to distribute the pressure. (See XIII.) In this case a 16 by 14 by 1¼-in CAST-IRON PLATE is used, uces the pressure on the brickwork to 82½ lb per sq in.

Fig. 8. This joint might be made in the manner described above, ype shown in Fig. 9 is used. The thickness of the plate is usually by the thickness required at *Y* to give the HOOK the proper strength.



Detail of Joint 1, Fig. 3

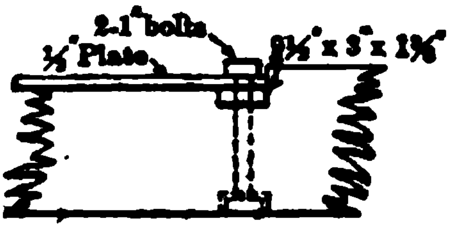


Fig. 9A. Alternate Detail of Joint 1, Fig. 3

ıractically takes one-half the horizontal component of the stress in ıhich is the stress in the bottom chord in this case) as the bolts are rely to keep the parts in place. The metal bears against the end- each inch in depth of the notch, the fibers carry 9½ × 1 100 = 10 450 lb, e notch is ½ (27 350/10 450) = 1.31 in deep, say 1⅜ in. Considering a WROUGHT-IRON CANTILEVER, 1⅜ in long and uniformly loaded) per sq in, the thick- l from the expression 2 = ⅙(12 000 × 1 × t^2), in. The nearest e is a thickness of length of the bot- necessary to take from the hook in shear is 9½ × 100 r in, or, 13 675/950 all. The inclination at *H* with the ver- about 36°, and the sure on this surface — 200) (³⁶⁄₉₀)² = 344 350/(344 × 9½) = 8.3 in, which is the required depth of the cut. s are confined by the plate, one-half this value, or 4¼ in, approx- ed. The bolts, *Z*, are two ⅞-in bolts. There should be two efully placed so that the hook bears against them. There is a y for the hook to straighten and hence 1-in bolts are used. n of the plate in tension is evidently greatly in excess of that

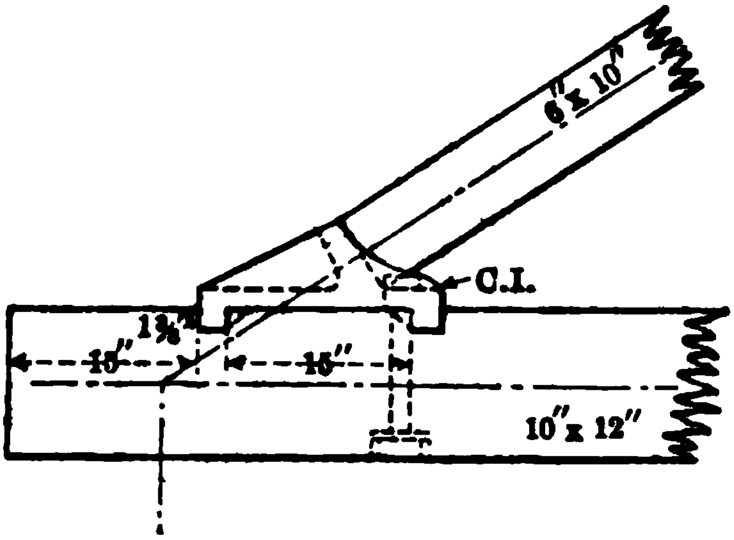


Fig. 9B. Alternate Detail of Joint 1, Fig. 3

. 8. A better detail at *Y*, Fig. 9, is shown in Fig. 9A. It is fore that one-half the tension is taken by the notch at *Y*. The

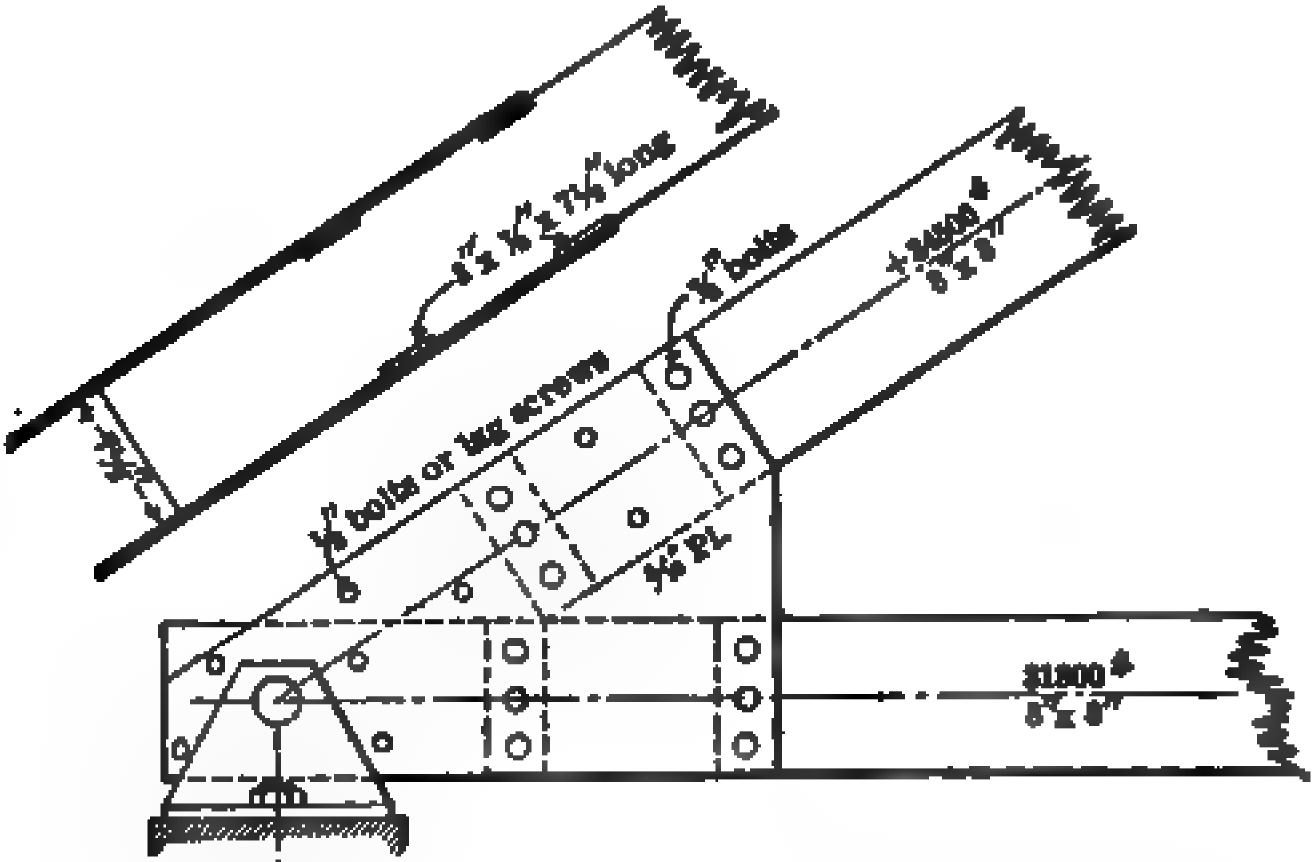


Fig. 9c. Detail of Joint 1, of Truss Similar to Fig. 1

lepth of the notch is 1¾ in and the size of the metal block, 1¾ by 3 by 9½
The bolts are assumed to have a close fit in the block and in the plate and be
carry the stress in single she
At 10 000 lb per sq
the area of the bolts is
(27 350/10 000) = 1.37 sq
requiring two 1-in steel b
(Table III, page 419). T
thickness of the steel pl
necessary to give suffici
bearing against the bolts
½ (13 675) ÷ 20 000 × 1, or
.34 in. A ½-in plate is the
fore ample.

Joint 1, Fig. 8. An ordin
CAST-IRON ANGLE-BLOCK
be used in this particular c
as shown in Fig. 9B.

Other Details for Joint **Fig. 3.** Another design
this joint, but for anot
truss, is shown in Fig.
The rafter and bottom ch
are of long-leaf yellow
and the metal parts of s
The stresses are transm
through 3 by ¾-in plat
bearing against the end-
of the wood, and from t
plates to the SIDE PL
through the bolts in bending. The side plates should be drawn up ag
the wood by LAG-SCREWS, as shown, to prevent buckling when in compres

) shows a good application of the CAST-IRON ANGLE-BLOCK used in the f a blacksmith-shop of the Boston & Maine Railroad Company. The and shearing values are provided for principally by a tenon on the into the bottom chord as indicated by the dotted lines.

, Fig. 1. Where a brace abuts against a rafter, as in this joint, one cut d of the brace should bisect the angle made between the brace and the d the second cut should be at right angles to this, as shown in Fig. 10. s then set in a notch or mortise to keep the brace in place and to transressure to the rafter. The purlin may be supported by a 3-in plank, as

Fig. 10A. Purlin-connection. Purlins on Top of Truss-chord

Detail of Joint 2, Fig. 1, with Rod Added

'urlin-con th Steel-

Fig. 10C. Purlin-connection with Wooden Bearing-block

with Beam-hanger

g. 10. Some form of METAL HANGER, of the DUPLEX TYPE is ed. In the truss shown in Fig. 1, there is no vertical rod at this any trusses have a rod there, and one is therefore shown in Fig. 10. on top of the rafter must have sufficient area to transmit the stress the rafter. Other forms of purlin-connections are shown in Figs.

King-Rod Truss. Fig. 11 shows the joint at the top of a KINGith a DUPLEX HANGER to support the purlin. The wrought-iron for large trusses should extend along the top of each rafter a ance to permit its being fastened by LAG-SCREWS or BOLTS. Fig 12 used in place of the ROLLED PLATE.

Joint 3, Fig. 1. This should be made as shown in Fig. 13. The inclin[ed] side bisect the angle made between the two 6 by 6-in pieces. In place of t[he] cast-iron washer a wrought-iron or steel plate may be used.

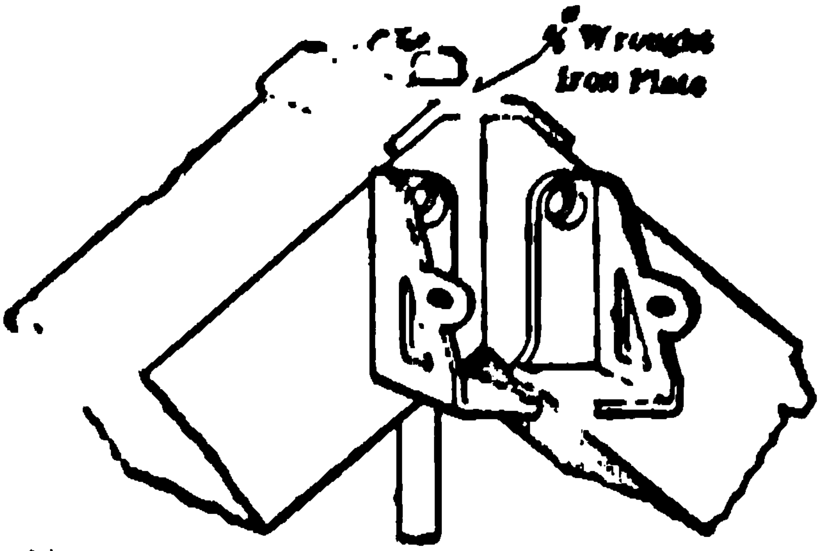


Fig. 11. Detail of Apex of King-rod Truss

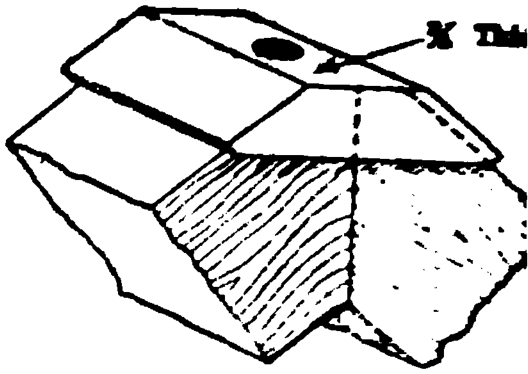


Fig. 12. Alternate Detail for Ape[x] of King-rod Truss

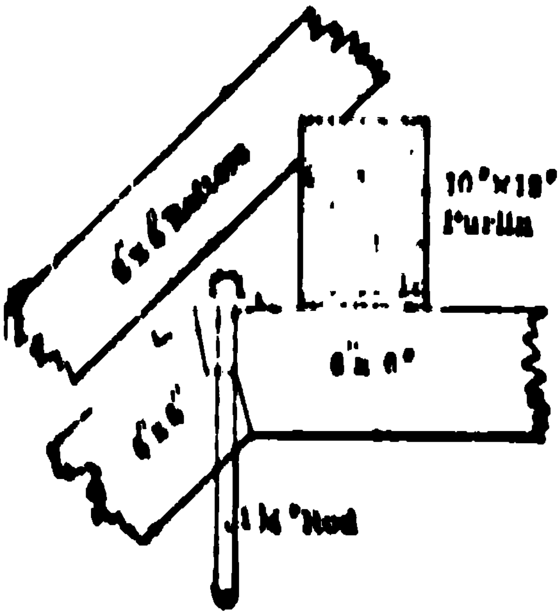


Fig. 13. Detail of Joint 3, Fig. 1

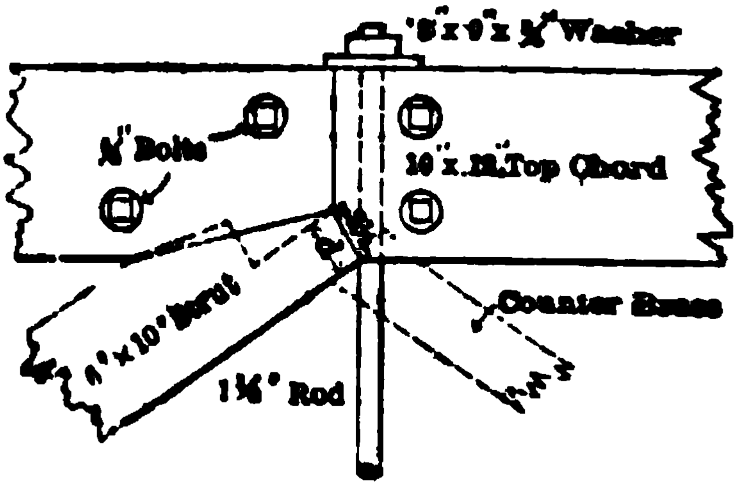


Fig. 14. Detail of Joint 2, Fig. 3

Joint 2, Fig. 3. One method of making the connections at this joint is show[n] in Fig. 14. The end-cut of the main brace is made as shown, the distance [...]

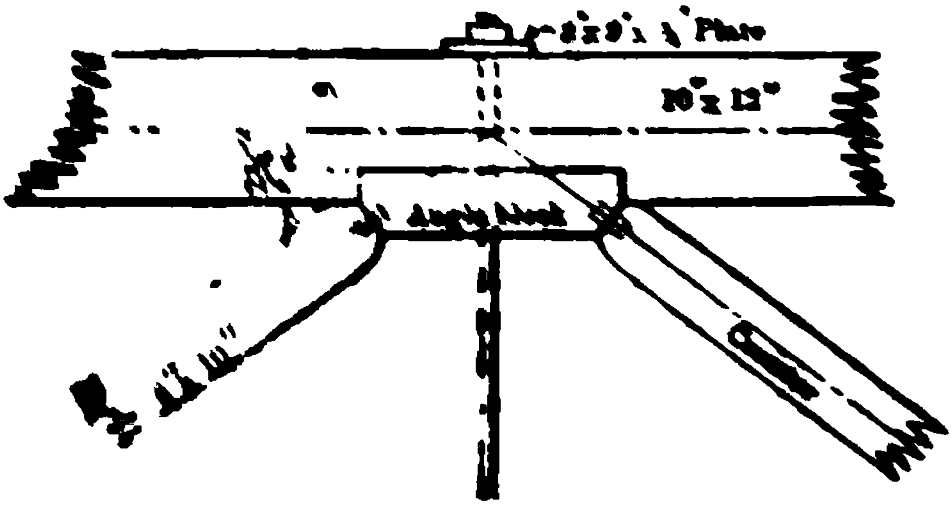


Fig. 15. Alternate Detail of Joint 2, Fig. 3

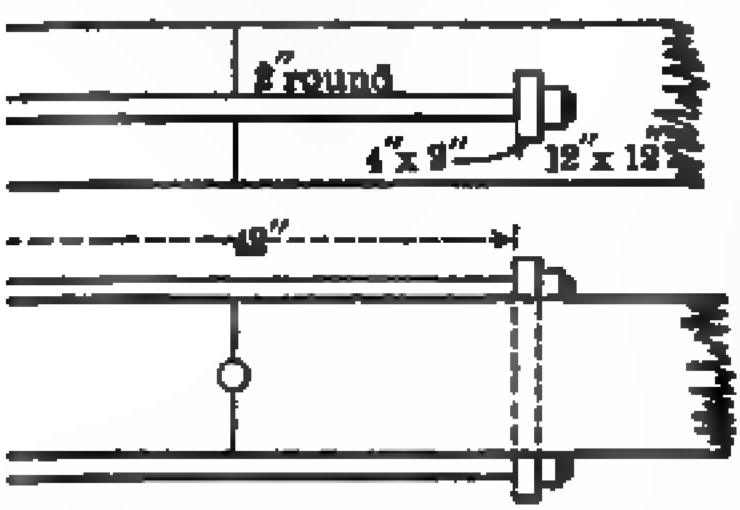
being determined by the necessary area of the inclined cut in the top chord. [illegible] per sq in. Then [illegible] required [illegible] a little greater than the depth of the brace. This

etail can only be used for the end-brace by making two notches as the dotted lines. A much better method is shown in Fig. 15, where BLOCK is used. The angle-block is made of very hard wood so that g of the brace is provided for, and it is notched into the chord a suffi- ent to transfer the horizontal component of the stress in the brace rd. A notch 1 in deep carries 1 100 × 9½ = 10 450 lb (Table I, page ace for a horizontal component of 27 350 lb (Fig. 4), the notch is in deep. This clearly braces should be inclined 5° with the horizontal, ward or weak details are ated. The vertical rod stress of 13 492 lb. The top of the chord transfers bearing across the grain. tress of 200 lb (Table I, the area is 67.4 sq in, m 8 by 9 by ¾-in

ig. 1. This is shown in Fig. 16, and the above discussions cover all design.

Since it is not economical and often impossible to procure timbers , or 30 ft in length, it is necessary to make one or more SPLICES s. The top-chord of a HOWE TRUSS is spliced by placing the tim- end, and by spiking or bolting on side planks to keep them in place. chord cannot be treated in this manner, as it is in tension.

ice or Tabled Fish-Plate of Wood. It is assumed that the l of the truss snown in Fig. 3 is to be spliced at the middle of the span. Fig. 17A shows this splice. It is assumed that the side pieces are of white pine. The total depth of the notches is 48 560 ÷ (1 100 × 11½) = d = 3.84 in (Table I, page 1138). Each notch, then, is about 2 in deep. The length of the table is l = ¼ [48 560 ÷ (100 × 11½)] = 21 in. The net thickness of each side



7. Splice of Bottom Chord of Truss

560)/(700 × 11½) = 3 in, without deducting anything for the two he chord-pieces have less than the required area because of the required; hence a 12 × 12-in timber is required if this form of The proper dimensions are shown in Fig. 17A.

se. Fig. 17 shows an old and very efficient form of splice, pro- splace the form shown in Fig. 17A.

Built-up Chord. The top chord, when BUILT UP of 2-in planks, igh spiking with two ¾-in bolts at the ends of each plank. The which is in tension, should be so arranged that the ends of one strand are well removed from the ends in other strands. The

middle strand of a BUILT-UP CHORD is completely cut away to permit the pass of the vertical rods. The strands should be thoroughly spiked, and bolted eve 2 ft, care being taken to see that the bolts do not come nearer than 5 from the end of any plank. While BUILT-UP MEMBERS are in favor with buil

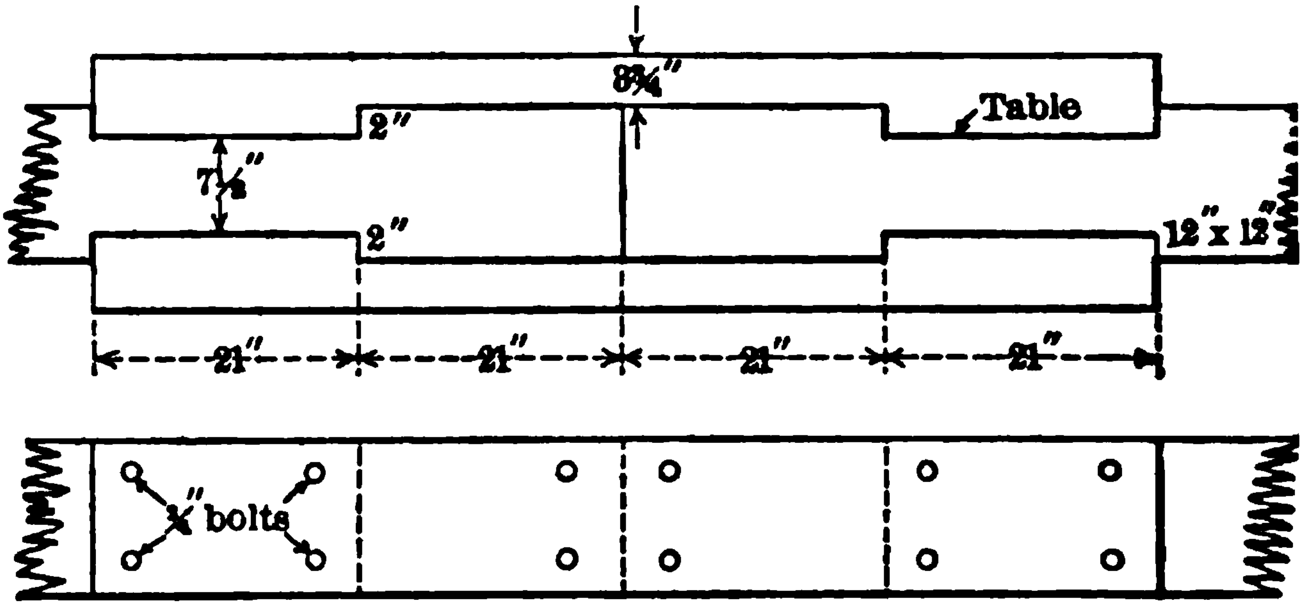


Fig. 17A. Alternate Detail for Splice of Bottom Chord

ers because the materials are readily obtained, yet for important structures t writer believes it is worth while to use a little more effort and pay a little m to get SOLID STICKS for truss-members.

Wall-Joint of Scissors Trusses. In SCISSORS TRUSSES the joint over t wall formed by the rafter and tie-beam should always be carefully proportion

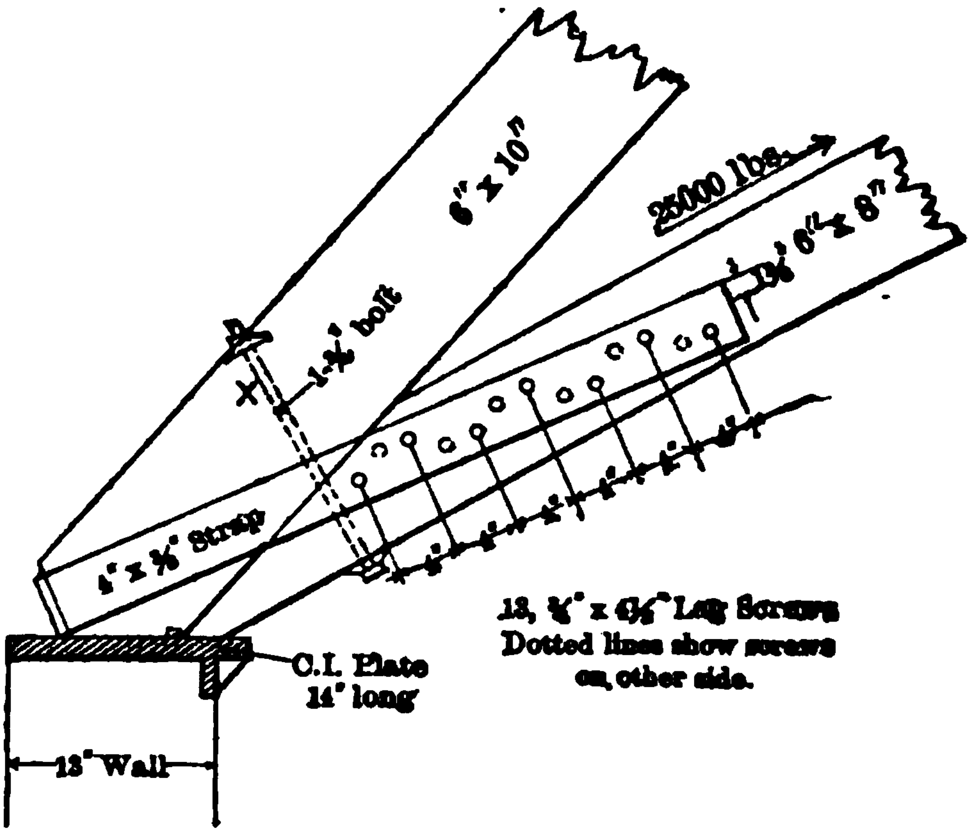


Fig. 18. Wall-joints for Scissors Trusses, Figs. 24 to 27, Chapter XXVI

to the stresses; otherwise the joint is liable to open and the wall to be push out. Much greater strength is required in this joint than in the wall-joint o KING-ROD TRUSS of the same span, because the stresses in a SCISSORS TRUSS usually at least twice and sometimes three or four times as great as in a tru with a horizontal tie-beam. For a SCISSORS TRUSS built of planks, as in Fig.

t through the center of each joint, with as many spikes as can be l ordinarily give sufficient strength. For trusses like those shown in 27 of Chapter XXVI, one of the best methods of making the wall-s the roof is quite flat, is that shown in Fig. 18, which is the detail l joint where the stress in the tie-beam was 25 000 lb. It should be t the WROUGHT-IRON STRAP is secured to the tie by LAG-SCREWS in-OLTS. It is practically impossible to bolt a strap to each side of a to get a good bearing for all of the bolts, owing to the difficulty in holes straight; and if the holes are bored a little large, some bolts on the wood and some may not. With LAG-SCREWS each screw is et a good bearing in the wood. The holes in the two sides of the , of course, be staggered, so that they will not come opposite each e net sectional area of the strap should at least be equal to the stress eam divided by 2 × 12 000 (Table I, page 1138). The number of , for both sides, is found by dividing the stress in the tie-beam by ce of one screw. For the safe resistance of LAG-SCREWS used in this lues given in Table V are recommended. In the joint shown in Fig. ss in the tie-beam is 25 000 lb, and the wood is Douglas fir. The therefore, require a sectional area in the strap of ½ (25 000)/12 000 = d twenty-three ¾-in lag-screws. Only thirteen are shown in Fig. 18.

Safe Resistance of Mild-Steel Lag-Screws When Used as in Fig. 18

crew in es	Safe resistance in pounds				Minimum thickness of strap in inches
length	Oak	White pine	Douglas fir	Long-leaf pine	
3½	288	255	267	288	¼
4	512	454	474	512	¼
4	800	709	741	800	5/16
4½	1 153	1 022	1 067	1 153	5/16
5	1 569	1 391	1 453	1 569	⅜

l upon experiments made (1915–1916) by Professor H. A. Thomas.

ickness of ⅜ in, the width of the strap necessary to give a sectional q in is 1.05/.375, or about 3 in. To this should be added the diam-ag-screw to obtain the working width. Thus 3 + ¾ = 3¾ in. The 4 by ⅜ in in cross-section, as some additional strength is obtained by , which it is necessary to insert to hold the timbers together while eing raised into position, and also to bring them tightly together the strap. Fig. 19 shows another method of making this joint e used with advantage when the inclination of the rafter is less ne advantage in using this truss is that if it is erected ONE PIECE e tie-beams may be put up first, thus providing a SEAT to receive The strap prevents the end of the rafter from springing up. The he bolt should be proportioned to the horizontal component of the rafter. Fig. 20 shows a good form of joint to use at joint 5 of Fig. XXVI, when it is desired to substitute a wooden tie for the rods 27. The sectional area of the strap and the number of lag-screws portioned by the rules given for Fig. 18.

Where iron or steel rods are used in wooden trusses, washers are der the heads and nuts to properly distribute the loads on the dimensions of the washers are determined by the allowable bear-

ing pressure on the wood and the magnitudes of the loads. Table VI gives th allowable loads which can be transmitted by standard round cast washers an rectangular washers bearing across the wood fibers. Table VII gives the di mension of standard round cast washers. The bearing areas of these washe

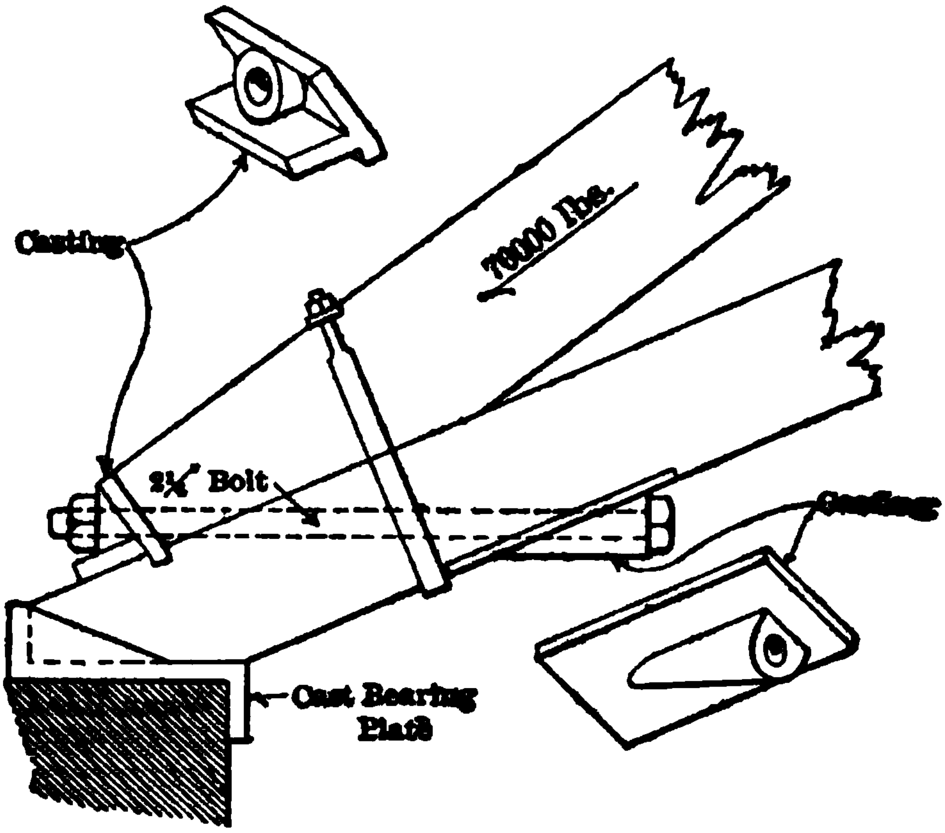


Fig. 19. Alternate Detail for Fig. 18

are too small for use on the softer woods and, therefore, except when the rod are small, it is better to use rectangular washers of iron or steel plate. Very large washers should be cast, and should have the form shown in Fig. 20A. The use of the ribs gives the required strength and saves considerable material

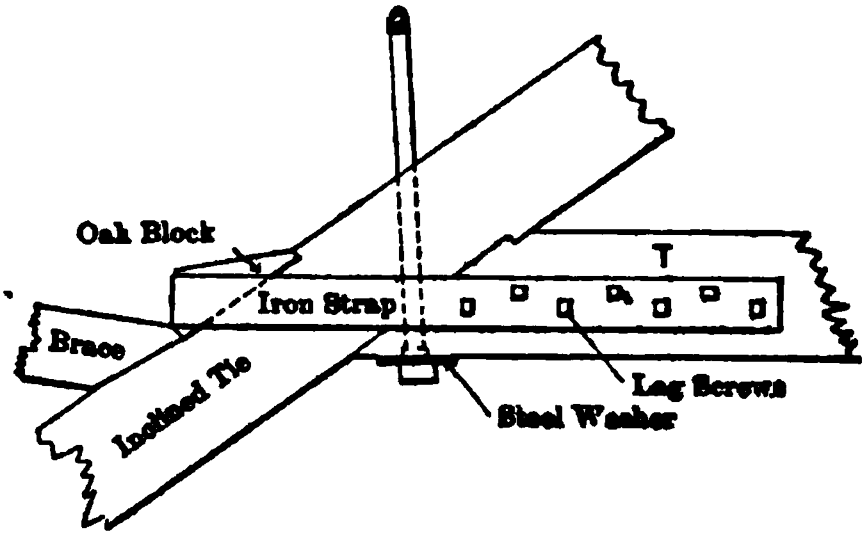


Fig. 20. Detail of Joint 5, Fig. 27, Chapter XXVI

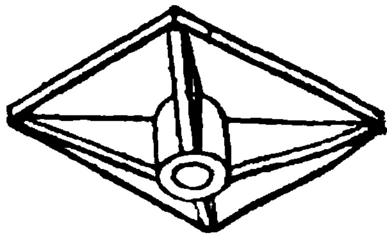


Fig. 20A. Cast-iron Washer with Brackets

Thickness of Rectangular Steel-Plate Washers. The thickness of rec tangular steel-plate washers can be found from the following formulas in which l is the distance from the edge of the plate to the nut and t the thickness of the plate. When used

On white oak....................................	$l = 3.4\,t$
On white pine....................................	$l = 5.2\,t$
On long-leaf yellow pine....................................	$l = 3.9\,t$
On short-leaf yellow pine....................................	$l = 4.6\,t$

Safe Bearing Resistance of Cast-iron Washers, in Pounds

Round washers

Area,* sq in	White pine, lb	Short-leaf yellow pine, lb	Long-leaf yellow pine, lb	White oak, lb
5.16	1 030	1 290	1 810	2 580
6.69	1 340	1 670	2 340	3 350
7.78	1 560	1 950	2 720	3 890
10.4	2 080	2 600	3 640	5 200
11.7	2 340	2 930	4 100	5 850
16.6	3 320	4 150	5 810	8 300
26.9	5 380	6 730	9 420	13 500
28.6	5 720	7 150	10 000	14 300
38.5	7 700	9 630	13 500	19 300
49.9	9 980	12 500	17 500	25 000
62.8	12 600	15 700	22 000	31 400
77.1	15 400	19 300	27 000	38 600
92.9	18 600	23 200	32 500	46 500
110.2	22 000	27 600	38 600	55 100

Rectangular washers

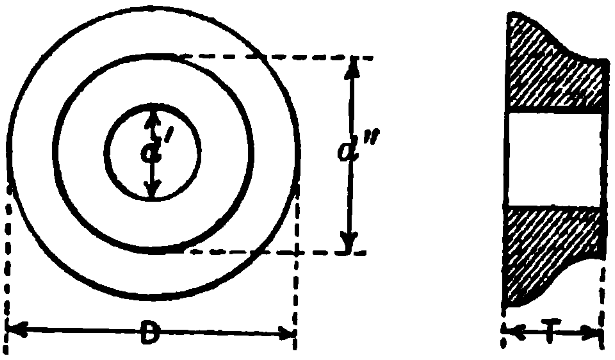
24	4 800	6 000	8 400	12 000
32	6 400	8 000	11 200	16 000
36	7 200	9 000	12 600	18 000
42	8 400	10 500	14 700	21 000
48	9 600	12 000	16 800	24 000
54	10 800	13 500	18 900	27 000
60	12 000	15 000	21 000	30 000
64	12 800	16 000	22 400	32 000
72	14 400	18 000	25 200	36 000
80	16 000	20 000	28 000	40 000
96	19 200	24 000	33 600	48 000
100	20 000	25 000	35 000	50 000
110	22 000	27 500	38 500	55 000
120	24 000	30 000	42 000	60 000
40	28 000	35 000	49 000	70 000
44	28 800	36 000	50 400	72 000
68	33 600	42 000	58 800	84 000
92	38 400	48 000	67 200	96 000
96	39 200	49 000	68 600	98 000
24	44 800	56 000	78 400	112 000
r sq in	200	250	350	500

bearing on the wood are given for round washers. For rectangula
a is given, no allowance being made for holes.

other forms of connections are in use and their proper desig
at the methods explained in Chapter XII and in this chapte
owed. All details are not suitable for all cases and the de
nmon sense in the selection of the PARTICULAR TYPE to b
m. Wood is very variable in its properties and consequentl
AFETY are used for certain kinds of stress and smaller facto

for others. Heavy trusses, in which the sizes of the members are selected cording to the magnitudes of the stresses, should be very carefully worked in every detail, while small trusses with large excess of material do not dem as much care.

Table VII. Proportions of Standard Cast-Iron Washers



Diam of bolt, d in	D in	d'' in	d' in	T in	Weight, lb	Bearing area, sq in
½	2⅝	1¾	9/16	⅝	½	5.16
⅝	3	1⅞	11/16	¾	¾	6.69
¾	3¼	2⅛	13/16	⅞	1¼	7.78
⅞	3¾	2½	15/16	⅞	1½	10.40
1	4	2¾	1 1/16	1⅛	2½	11.70
1⅛	4¾	2¾	1 3/16	1⅛	3	16.60
1¼	6	3	1 5/16	1⅜	5¾	26.90
1½	6¼	3¼	1⅝	1½	6	28.60
1¾	7¼	3¾	1⅞	1¾	9½	38.50
2	8¼	4¼	2⅛	2	17¼	49.90
2¼	9¼	4¾	2⅜	2¼	20	62.80
2½	10¼	5¼	2⅝	2½	27¼	77.10
2¾	11¼	5¾	2⅞	2¾	36	92.90
3	12¼	6¼	3⅛	3	46	110.20

For sizes not given, $D = 4d + \frac{1}{4}''$ $\quad d'' = 2d + \frac{1}{4}$
$d' = d + \frac{1}{8}$ $\quad T = d$

4. Joints of Steel Trusses

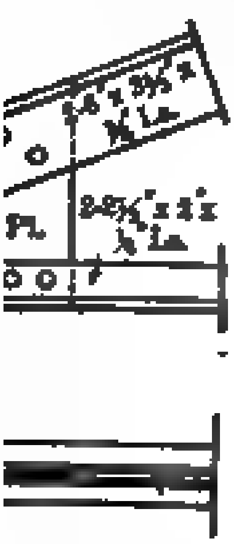
Trusses with Riveted Joints are usually made with ANGLES for the we members and generally for the chords, although the latter are sometimes ma of a pair of CHANNELS or of two ANGLES and a WEB-PLATE. The members a connected at the joints by means of GUSSET-PLATES, to which all of the membe are RIVETED. Typical examples of riveted joints in roof-trusses are shown Figs. 22 to 24E. When the rafter or chord has a WEB-PLATE, as in Fig. 23 the web-members are riveted to this plate and a GUSSET-PLATE is not requir except at the end-joint and apex, as shown in Figs. 23A and 23E. In order th there shall be no twisting, it is necessary to make the principal members of t truss DOUBLE, so that the gusset-plates can be riveted between them. Whe single angles are used for web-members and two such members come at a joint they should be riveted to opposite sides of the gusset-plates. For equ strength the thickness of the gusset-plate should be such that the BEARING the rivets equals the strength of the rivets in DOUBLE SHEAR, the thickness, ho ever, not exceeding the combined thickness of the two angles. Practical co siderations seldom make the gusset over ⅜ in thick for ordinary constructi

joints, which should be done to a scale of not less than 1 in
mbers should be arranged, when practicable, so that the lines
eir centers of gravity will coincide with the lines of the truss-
meet at a single point, as in Fig. 21. This is not always prac-
prin-
owed
sible.
, the
f the
con-
rious
ough
avity
The
s re-
mem-

Fig. 21. Riveted Truss-joint with Truss-diagram Lines

rmined according to the stress in that member, the resistance
g considered for both SHEARING and BEARING. The method
e number of rivets in a joint is explained in Chapter XII,
re clearly the application to truss-joints, the joints for the
g. 7 will be designed.

siderations, Truss of Fig. 7. It is assumed that the truss
three parts, making all the joints SHOP-RIVETED except those
ices at each end of the piece *EH*. All gusset-plates are to be
ll rivets ¾-in, except in the 2-in legs of angles, where ⅝-in
used. Since the bearing of a ⅝-in plate on a ¾-in rivet at
in (Table I, page 1138) is 5 630 lb, or at 18 000 lb per sq in
419) is 5 060 lb, and the resistance of the rivet in double shear,
lb (Table III, page 419), the number of rivets in all the joints
he bearing value. Only one leg of the angles will be connected



tail of Joint *A*,
ss, Fig. 7

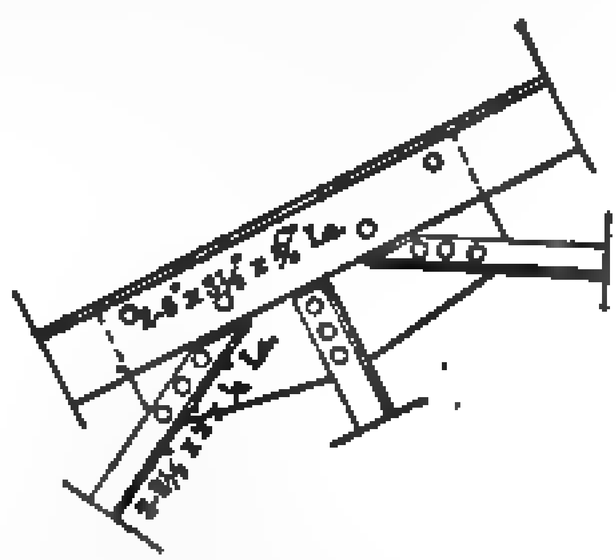


Fig. 22A. Detail of Joint *D*, Fink Truss, Fig. 7

ate as about 80% of the full strength of the angle is thereby
t less than three rivets are used. The use of HITCH-ANGLES
ing leg has but little influence in increasing the efficiency of the
o rivets may be considered the minimum number in any con-
ss of the unimportance of the member.

1. The top-chord stress is 23 500 lb, and if one rivet carries
e gusset-plate, five rivets will be required to carry this total

stress. In like manner four rivets are required for the bottom chord. T supporting force or the reaction is transferred to the gusset through the bott chord prolonged. In this case the reaction is about 8 800 lb which requires t rivets. Fig. 22 shows the arrangement of this joint at the expansion-end.

Joint D, Fig. 7. The web-members each require less than one rivet, but t or three should be used. Since the top-chord angle is continuous, the numbe rivets in it is determined by the difference between the two adjacent stres and the load of the purlin if it rests on the chord. Here again the number rivets required falls below the minimum number. Fig. 22A shows this joint.

Joint E, Fig. 7. The piece *CE* requires four rivets and the web-members t minimum number permissible. The piece *EH* requires, at 20 000 lb per sq

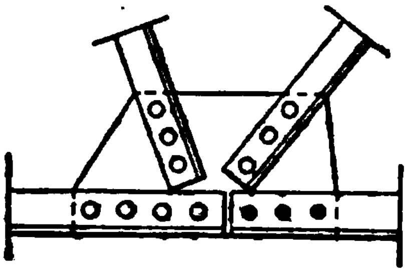


Fig. 22B. Detail of Joint *E*, Fink Truss, Fig. 7

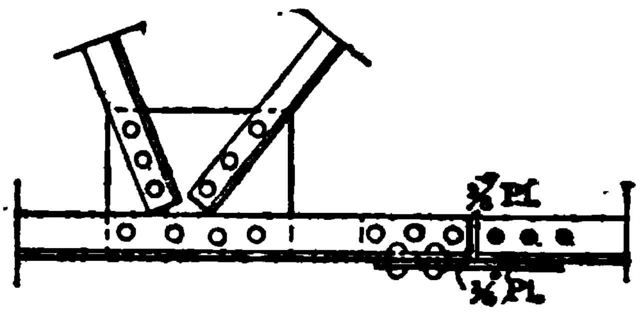


Fig. 22C. Detail of Joint *E* and Splice in *EH*, Fink Truss, Fig. 7

bearing value, 12 500/5 630 = 2.22 rivets; but as this connection is one to made IN THE FIELD, it is customary to increase the number 25%. This mak the required number three. Sometimes the outstanding legs are spliced to t member *CE* by a plate. Without doubt this increases the strength of the joi but it is doubtful if the increase in strength is enough to offset the extra co Fowler's specifications do not permit the piece *EH* to be connected to t gusset-plate. They specify that the connection shall be made upon the right *E*. This arrangement allows the use of a smaller gusset-plate at *E* which m be counterbalanced by the a ditional metal required for t splice beyond *E*. (Figs. 2 and 22C.)

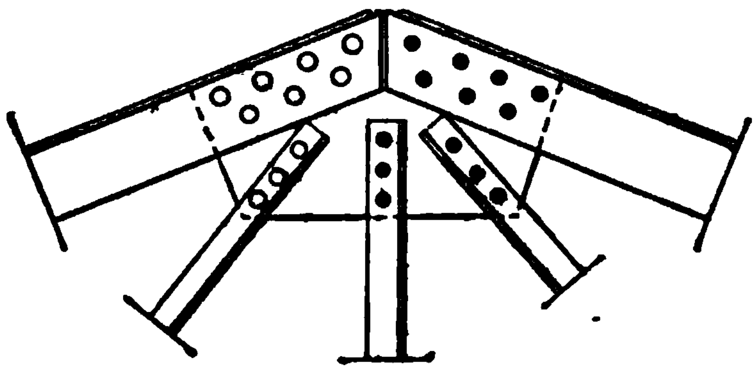


Fig. 22D. Detail of Joint *G*, Fink Truss, Fig. 7

Joint G, Fig. 7. The piec *BG* and *FG* are SHOP-RIVET to the gusset on one side a FIELD-RIVETED on the other. I order to make the joint sym metrical, the number of SHO RIVETS is made the same required for the FIELD-CONNECTION. In this case the top chord requires f rivets and the web-member three. Two rivets may be used in the sag-ti (Fig. 22D.)

Field-Connections. Bolts are often used instead of rivets for making FIEL CONNECTIONS. If the bolts fit the holes snugly, there is no serious objection t their use. In fact a good bolt is better than a poor rivet. For important wor however, bolts should not be used unless turned true to size and driven into tr holes. Open holes or holes for FIELD-RIVETS are indicated by BLACK CIRCLES.

Shop-Drawings. It is not advisable for the architect to make compl drawings for the steelwork. He should make what are usually designated GENERAL DRAWINGS. These are made to scale and give the general dimensi

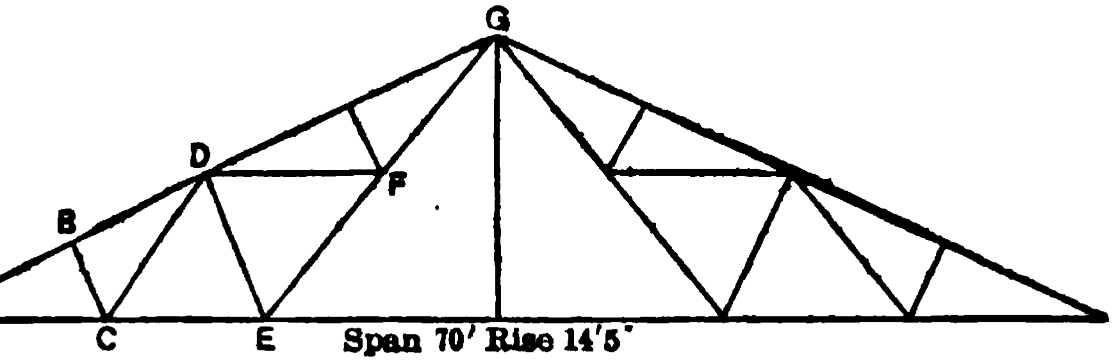


Fig. 23. Fink-truss Diagram. (See, also, Figs. 23A to 23E)

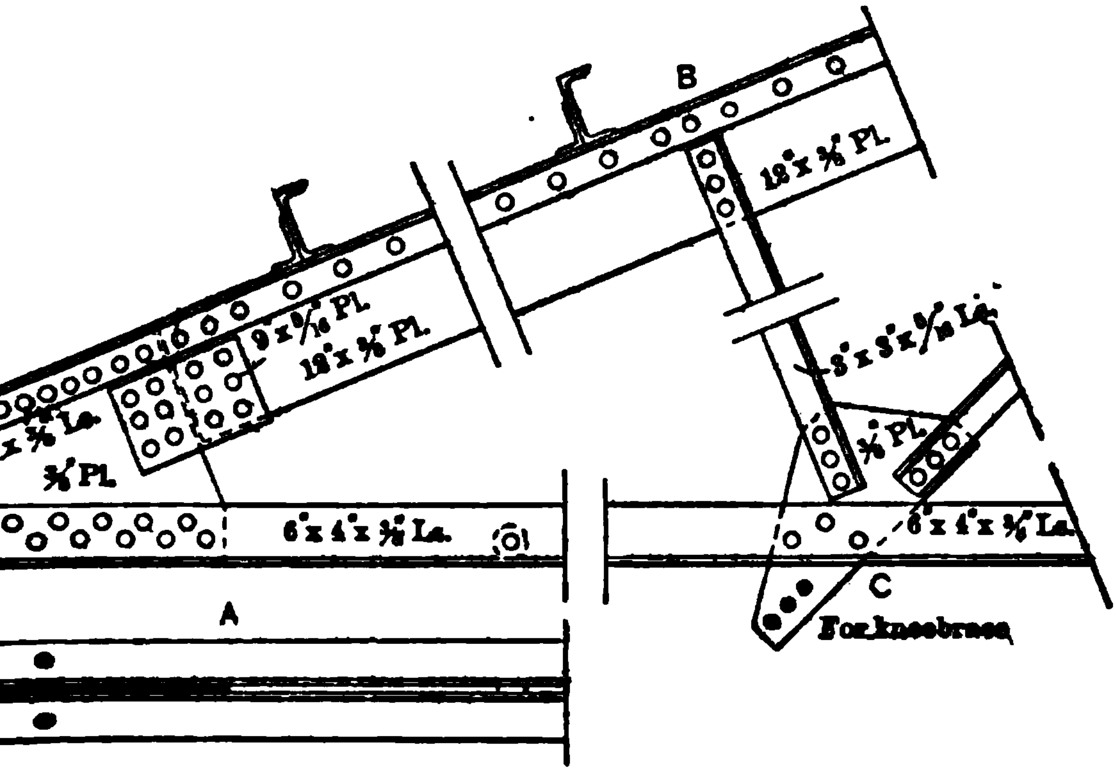


Fig. 23A. Detail of Joints *A*, *B* and *C* of Fig. 23

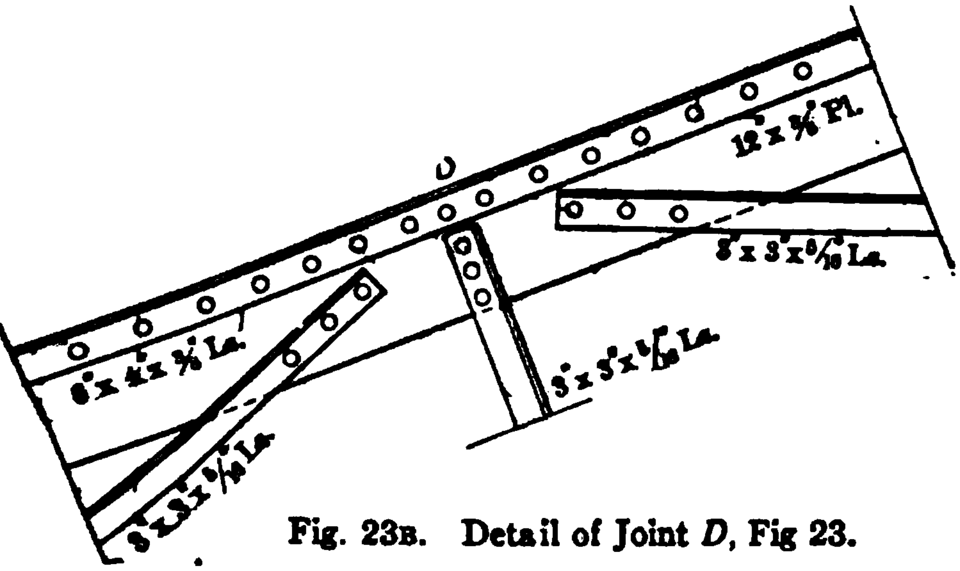


Fig. 23B. Detail of Joint *D*, Fig 23.

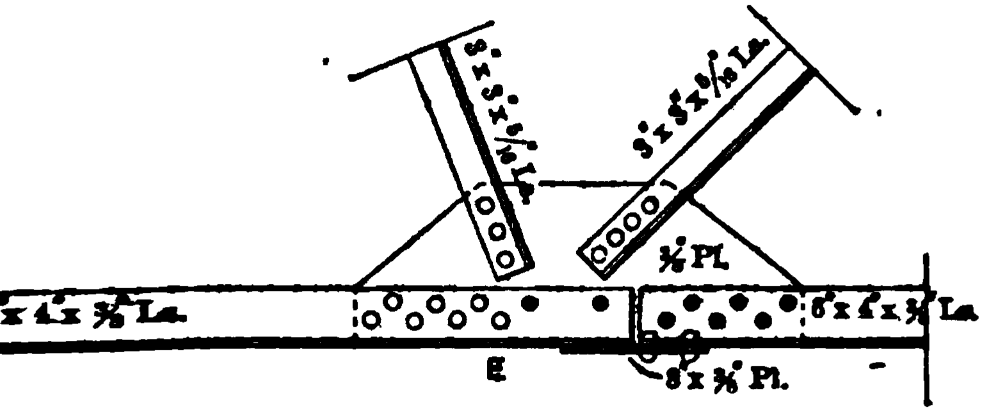


Fig. 23C. Detail of Joint *E*, Fig. 23

and sizes of the various members, and show each rivet approximately in its proper position. The manufacturer who fabricates the structure prefers to make his own SHOP-DRAWINGS to conform with his standards and methods.

Examples from Practice. Figs. 23 to 23E show the details of a modern shop-truss. These details were taken from the SHOP-DRAWINGS but with the rivet-spacing omitted. No metal under 5/16 in thickness, or rivets under 3/4 in

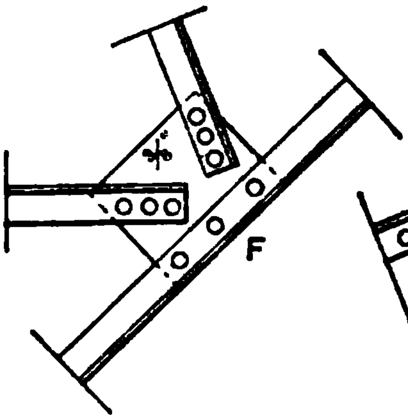


Fig. 23D. Detail of Joint *F*, Fig. 23

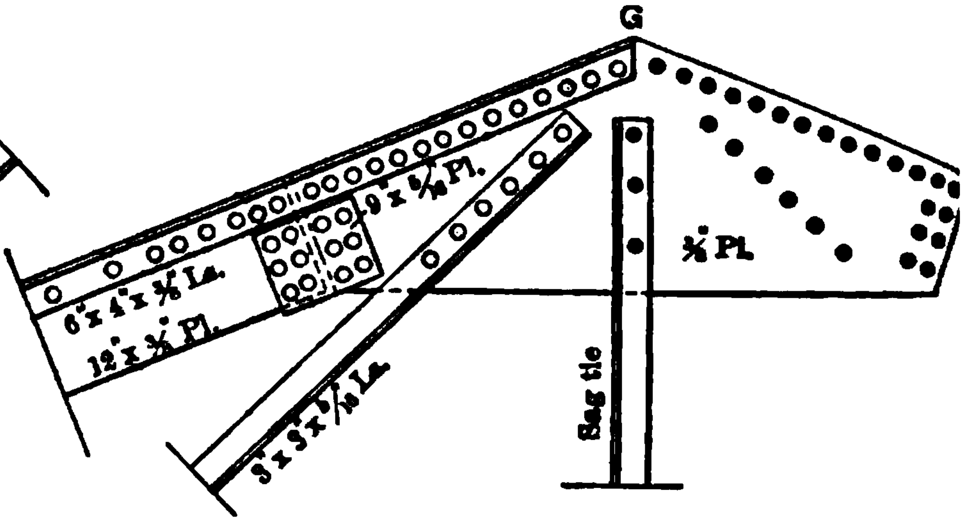


Fig. 23E. Detail of Joint *G*, Fig. 23

in diameter, are used. Another point may be mentioned in connection wit this truss; very few BEVEL-CUTS are made. The contrary appears to be th case for the details of the very light truss shown in Figs. 24 to 24E, in whic BEVEL-CUTS are made on the angles and more cuts than necessary on the gusse plates. These two trusses were designed by manufacturers widely separate

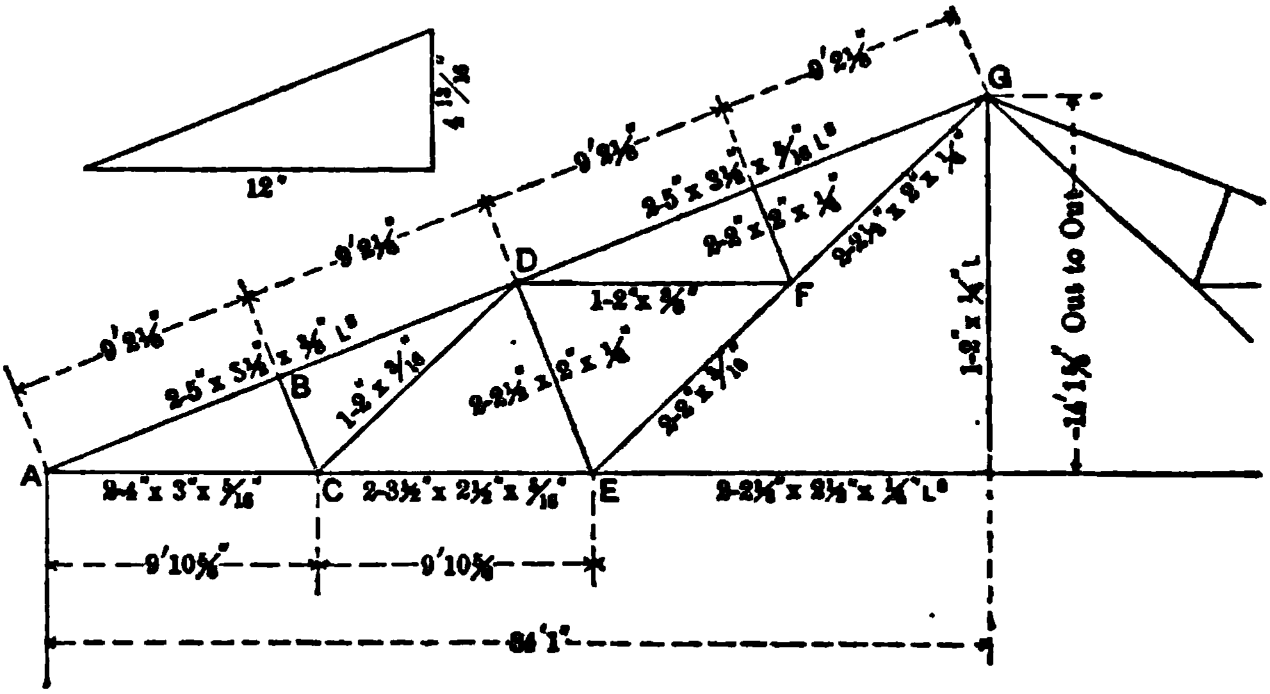


Fig. 24. Fink-truss Diagram. (See, also, Figs. 24A to 24F)

and are quite different in their details; and the variations emphasize the fa that the architect should not attempt to make SHOP-DRAWINGS.

Trusses with Knee-Braces. Fig. 25 represents joint 1 of Fig. 55, Chap XXVI, and was engraved from the WORKING DRAWING made by the New Jer Steel and Iron Company, Trenton, N. J. Another detail of a truss-connecti to a column is shown in Fig. 26. This was used in the template-shop roof-tru Ambridge plant of the American Bridge Company, Ambridge, Pa. Fig.

Fig. 24A. Detail of Joint A, Fig. 24.

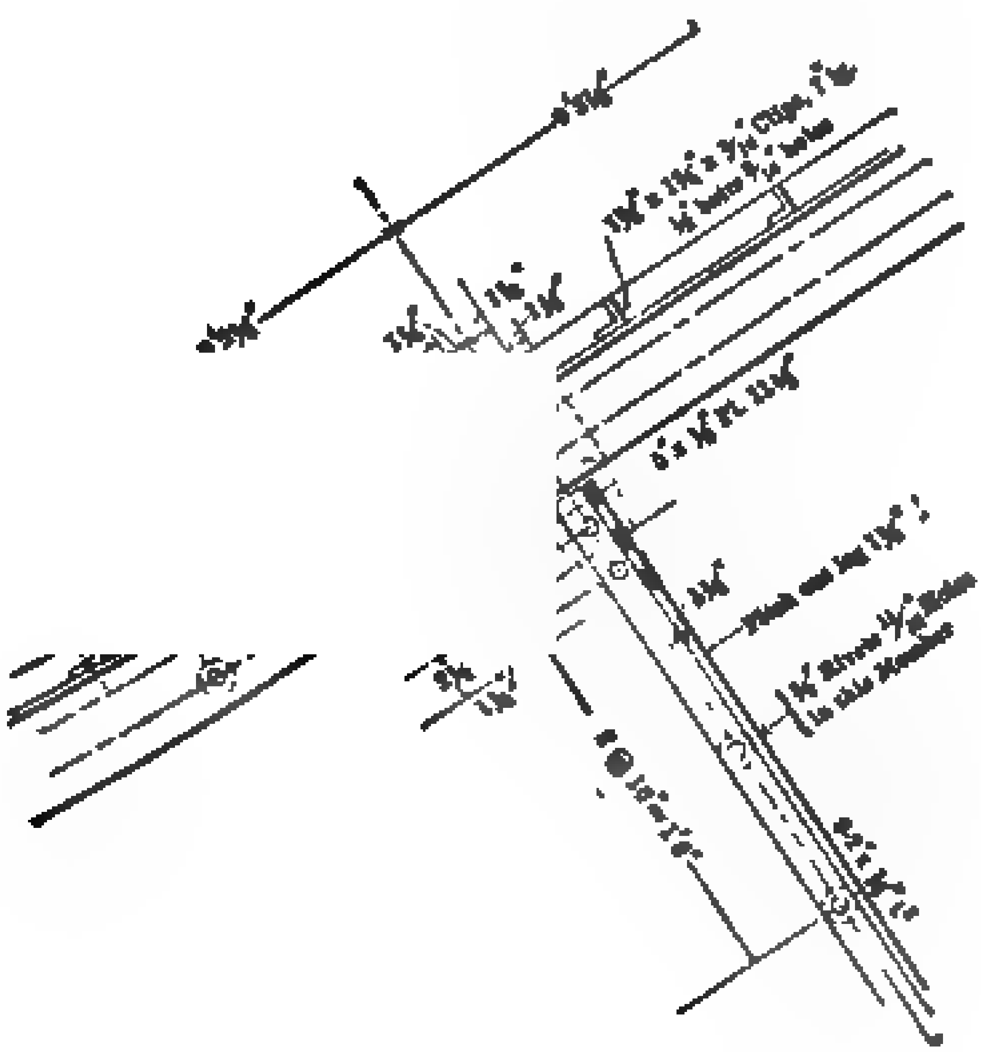


Fig. 24B. Detail of Joint B, Fig. 24.

Fig. 24c. Detail of Joint *D*, Fig. 24

Fig. 24d. Detail of Joint *F*, Fig. 24

Fig. 24x. Detail of Joint G, Fig. 24

g. 25. Detail of Joint 1, Fig. 55, Chapter XXVI

shows the wall-end of a small truss supported by brick walls. The intersecti of the STRESS-LINES is approximately in a point above the center of the suppo

Fig. 26. Connection of Steel Truss with Steel Columns

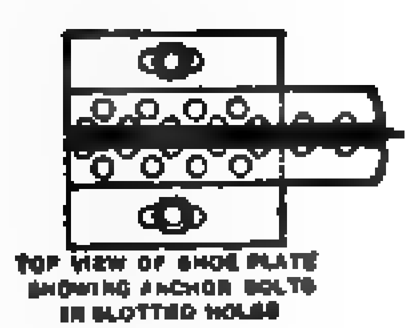


Fig. 27. Wall-end of Steel Truss. Support and Anchoring-details

This condition is seldom considered by architects. Usually it is possible witho any extra expense to satisfy this requirement and thereby to a great extent p vent unknown bending-stresses.

5. Purlins and Purlin-Connections

Where the roofing is supported directly on the PURLINS, as is gen-
ase in light steel roofs, the purlins and trusses are usually spaced so
ıer that SIMPLE ROLLED SHAPES may be used for the purlins. For

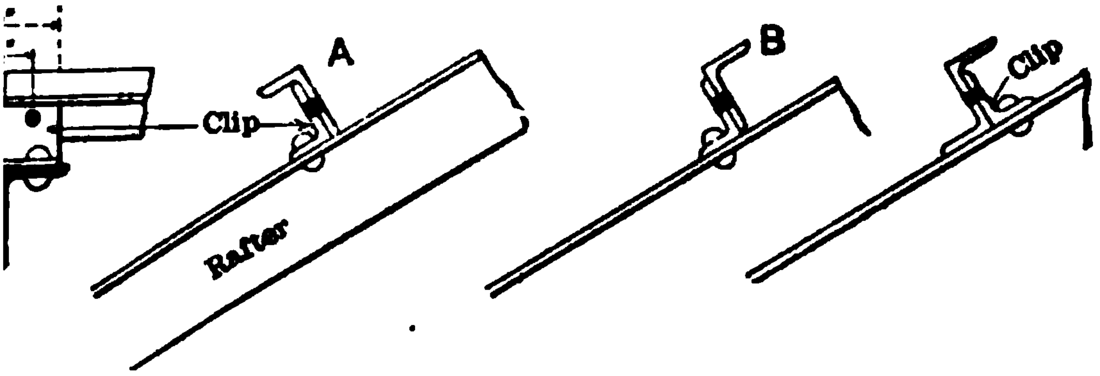
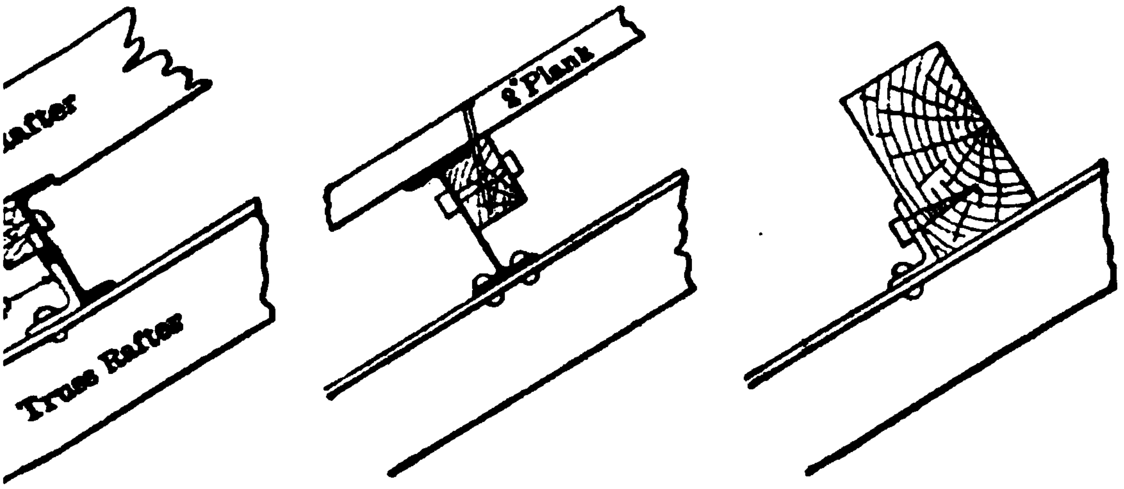


Fig. 28. Purlin-connections. Steel Clips, Angles and Z Bars



). Purlin-connections. Steel Sections with Wooden Nailing-strips

n trusses of from 8 to 10 ft, ANGLES are commonly used, and for
, Z BARS, CHANNELS and I BEAMS. WOODEN PURLINS are often used
ısses. If STEEL PURLINS support wooden rafters or plank roofing,
RIP of wood is bolted to the purlin, as
29. When the distance between purlins
ıre, a line of ⅝-in rods should run from
ıugh the purlins, to prevent them from
: plane of the roof. The purlin at the
ıe designed to take the vertical com-
stress in these rods.

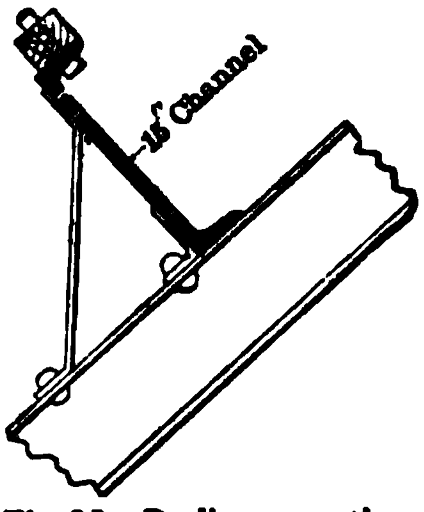


Fig. 30. Purlin-connection. Braced Channel

ınections. Figs. 28, 29 and 30 show a
ious methods of fastening the purlins to

Purlins. Fig. 31 shows the cross-
RECTANGULAR WOODEN PURLIN and of
ED STEEL SHAPES employed for purlins.
page 1170, when using wooden purlins
ır the stress in the outer fiber is true
ɜd with reference to the PRINCIPAL AXES of the section. Then,
ıl axis does not lie in the plane of the loading, the loading must
ıto two components, respectively parallel to the two principal
so, pages 573 and 593.)

Let S' = the fiber-stress with reference to the principal axis, AA, for the re tangle, 1–1 for the I beam and channel, and 4–4 for the angle and Z bar. M' the bending moment of the component of the load which lies in the plane p pendicular to the above axis. I' = the moment of inertia of the section wi

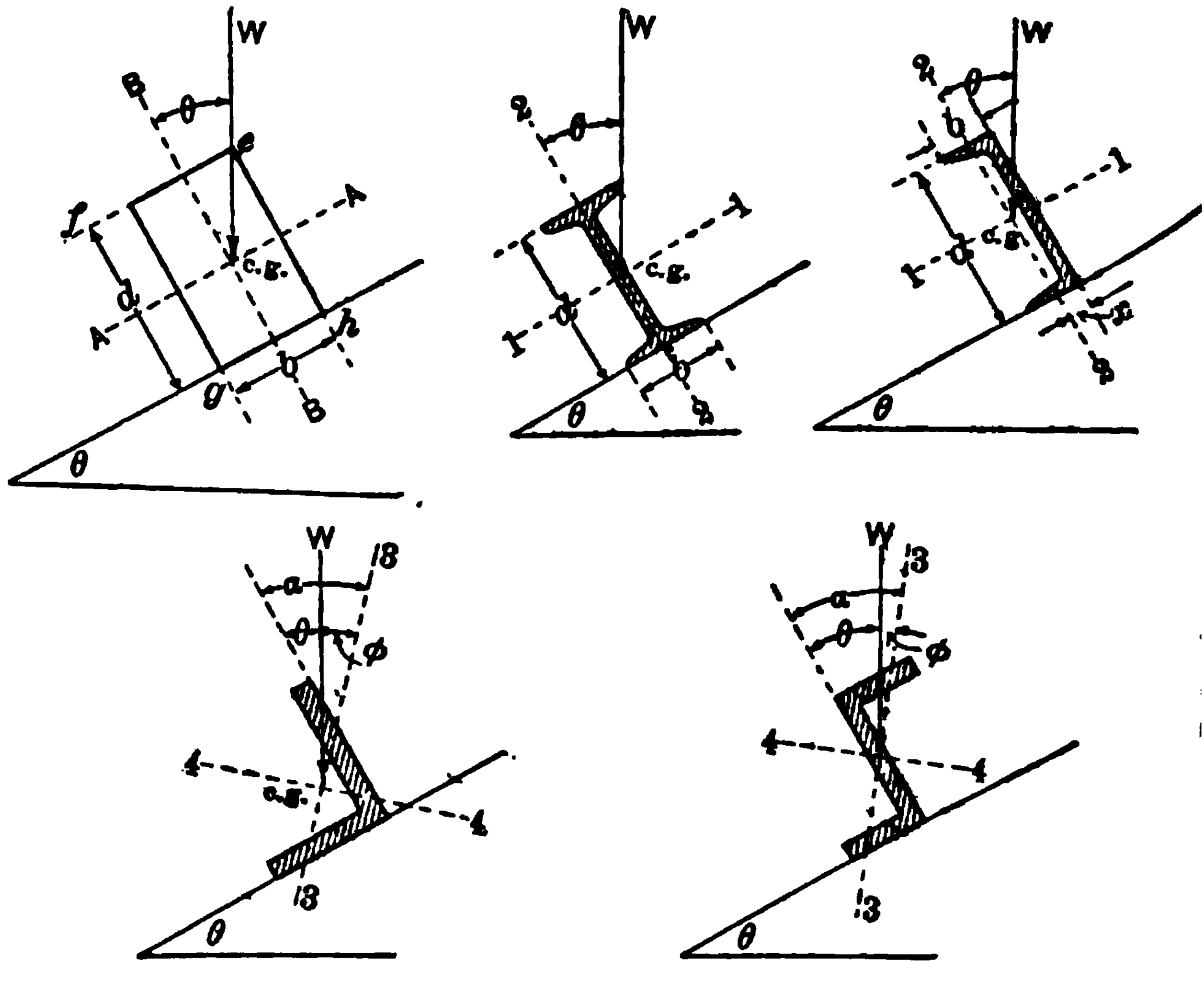


Fig. 31. Sections of Wooden and Steel Purlins

reference to the above axis. c' = the distance of any selected fiber from above axis. For the other principal axis use S'', M'', I'' and C''; then if the resultant fiber-stress,

$$S = S' + S'' = M'c'/I' + M''c''/I''$$

For the rectangle,

$$S = S' + S'' = 6\,M'/bd^2 + 6\,M''/b^2d$$

For the channel and I beam,

$$S = S' + S'' = M'd/2\,I_{1-1} + M''b/2\,I_{2-2}$$

For the angle and Z bar,

$$S = S' + S'' = M'c'/I_{4-4} + M''c''I_{3-3}$$

The application of these formulas offers no difficulties except in the c of ANGLES and Z BARS. For the other forms, the values of I and c are giv the tables of properties of the sections (Chapter X). The locations of the cipal axes for the Z bars and angles are also given in the tables, but the v of c are not given for any of the fibers. The easiest way to get the value in any particular case is to draw the section of the angle or Z bar full size, la the principal axes and then measure the actual distances, c.

CHAPTER XXIX

WIND-BRACING OF TALL BUILDINGS

By

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OF

PURDY & HENDERSON, INC., CIVIL ENGINEERS

1. Data for Wind-Pressure. Building Laws

ildings of Modern Construction, that is, buildings with skeleton light curtain walls or filler walls, require that resistance to wind-considered with care. The proportions of a building and the arrange-trength of the walls determine to what extent special bracing must l. Building ordinances in the larger cities usually require consider-stated wind-pressure. Where such ordinances do not definitely fix d pressure, a unit force of 30 lb per sq ft of surface is generally proper and adequate. (See, also, page 150.)

Laws. The following are extracts * from the building ordinances of City, Chicago, Philadelphia and Baltimore with reference to wind-

New York (1917)

CONSIDERED. All buildings over 150 ft in height and all buildings buildings in which the height is more than four times the minimum limension, shall be designed to resist a horizontal wind pressure every square foot of exposed surface measured from the ground to e structure, including roof, allowing for wind in any direction.

Y. The overturning moment due to wind pressure shall not exceed of the moment of stability of the structure, unless the structure is hored to the foundation. Anchors shall be of sufficient strength ry the excess overturning moment, without exceeding the working bed.

LE STRESSES. When the stress in any member due to wind does per cent of the stress due to live and dead loads, it may be neg-n such stress exceeds 50 per cent of the stress due to live and dead rking stresses prescribed may be increased by 50 per cent in design-bers to resist the combined stresses."

Chicago (1915)

ngs and structures shall be designed to resist a horizontal wind enty pounds per square foot for every square foot of exposed sur-case shall the overturning moment due to wind pressure exceed er cent of the amount of stability of the structure due to the dead

produced by wind forces combined with those from live and unit stress may be increased fifty per cent over those given above; shall not be less than required if wind forces be neglected."

rally. Form in general not edited or changed. Some paragraph-captions ate editor.

Philadelphia (1915)

WIND PRESSURE. "In all buildings allowances shall be made for wind pr sure, which shall not be figured at less than thirty pounds per square foot elevation where erected in open spaces or upon wharves. In high buildin erected in built-up districts, the wind pressure shall not be figured for less th twenty-five pounds at tenth story, two and one-half pounds less on each succe ing lower story, and two and one-half pounds additional on each succeed upper story, to a maximum of thirty-five pounds at fourteenth story and abo

WIND BRACING. "Wind bracing may be provided by making the connect joint between girders and columns sufficient for the vertical load as well as bending due to side pressure; or brackets may be placed at this joint, prop tioned for the side pressure; or diagonal bracing may be placed between colum proportioned to transfer the shear of the side pressure to the footings.

BASE OF COLUMN MUST BE ANCHORED. Where buildings are narrow tall, so that the overturning due to wind is more than the down pressure of unloaded building, the base of column must be anchored down to a suffici foundation to counteract this upward strain." *

Baltimore (1914)

WIND PRESSURE. "All new buildings exposed to wind shall be made str enough to resist a horizontal wind pressure in any direction of thirty pounds square foot of exposed surface, measuring the entire height of the building.

CALCULATION OF. "The additional loads caused by the wind pressure u beams, girders, walls and columns must be determined by calculation and ad to other loads for such members, as provided for in Section 19 of this Artic

SPECIAL BRACING. "Special bracing shall be employed wherever neces to resist the distorting effect of the wind pressure.

OVERTURNING MOMENT. "In no case shall the overturning moment du the wind pressure exceed fifty per cent of the moment of the stability of structure."

Magnitude of Unit Stresses Used for Wind-Pressure. As the ab extracts indicate, it is generally considered proper to use HIGH UNIT STRES when allowing for wind-pressure. The practice is based on the assumption t the HIGHEST UNIT WIND-PRESSURE will occur very infrequently and that duration usually will be limited to a very few moments. It should be no that the combined stresses due to wind-loads and dead and live loads sho not exceed ordinary stresses by more than 50%. If stresses developed by wind alone do not exceed 50% of those due to dead and live loads, they may neglected.

2. Conditions Determining or Affecting Wind-Bracing

Construction which Resists Wind-Pressure. The dead weight of a bu ing, the exterior walls, the interior partitions and the ordinary connection beams to columns, all aid in resisting wind-pressure, but to a degree whic not determinable in any exact way; and these factors vary greatly, also different buildings. Any allowance for these factors must be largely a quest of pure guesswork, or it may be judgment, based on the resistance which o buildings have offered when no special bracing was provided. It is there best to make special bracing take care of all, or very nearly all, of an ASSU MAXIMUM PRESSURE, when the building under consideration is unusually ligh construction, or when its proportions are such as to make resistance to w pressure a prime consideration.

* Stress is meant.

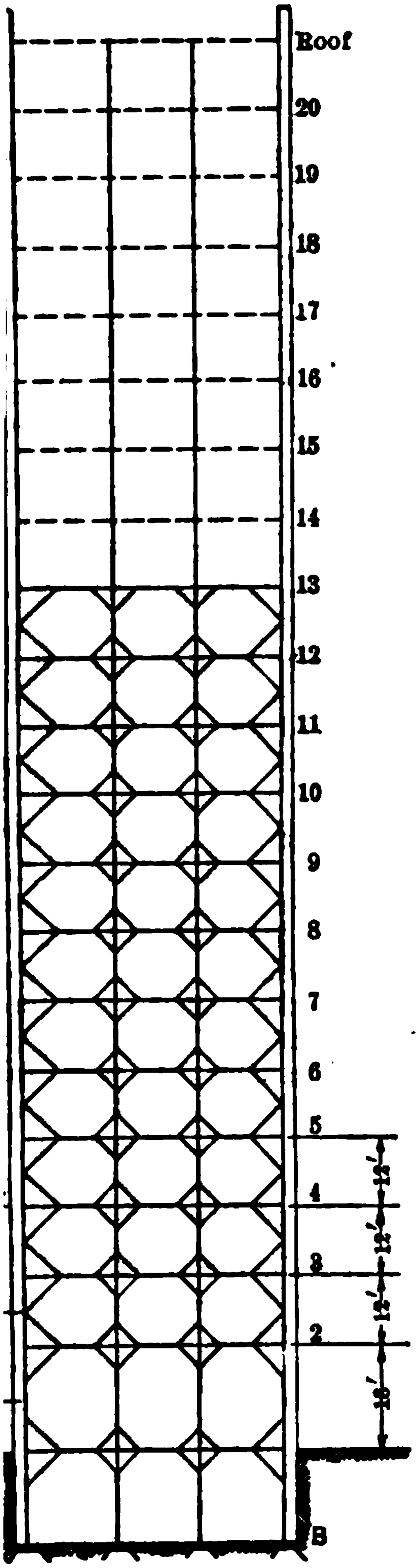
† This refers to a section of the Baltimore building laws.

Width as Affecting Wind-Pressure. It is generally safe to ssure in structural designs for buildings ten stories or less in height, where the average width is not less than one-third the height. It is also usual to omit special provision for wind-bracing in higher buildings where the width is two-thirds the height, or more. The writer believes the above approximations represent conservative practice, so far as general rules are possible.

Dead Load as Affecting Wind-Pressure. A building should not be so proportioned that the OVERTURNING MOMENT of a wind-pressure of 30 lb per sq ft exceeds 75% of the available RESISTING MOMENT of the dead load. If necessary, the columns should be anchored to the foundations.

3. General Theory of Wind-Bracing

Buildings Considered as Cantilevers. Buildings are usually considered to resist wind as



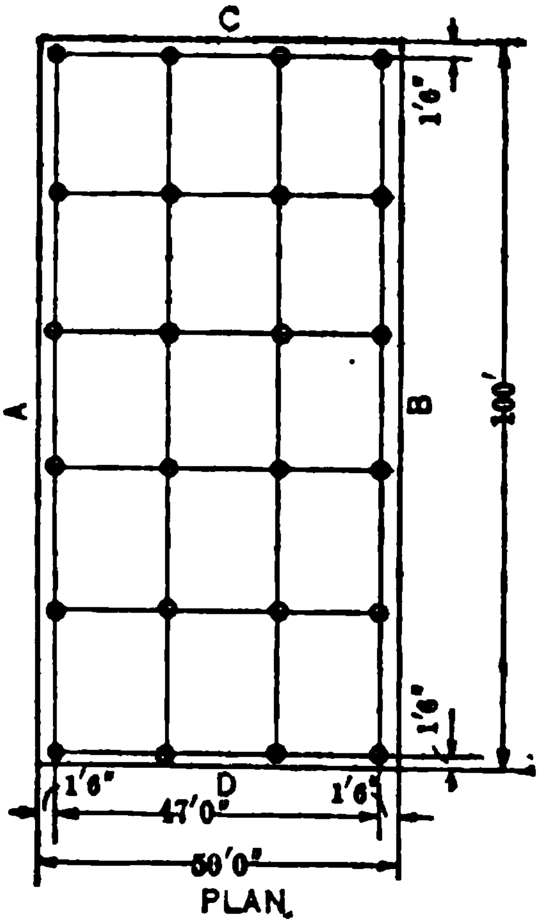


Fig. 1. Section and Plan of Wind-braced Building

RDERS or trusses, planted in the earth. Assuming a building dimensions shown in section and plan in Fig. 1. with a wind-

pressure against side A, the walls A and B, together with the colum beams, etc., in these walls, are the FLANGES of the girders. Walls C and with their framing, together with other intermediate lines of vertical frami form the WEB of the cantilever and transmit the vertical shears. Steel bra ing in horizontal planes is seldom necessary, as ordinary floor-constructio are generally sufficient to transmit wind-loads to the vertical bracing. In so cases, however, it is necessary to add steel bracing in the floors. Such a c is found in the tower of the new Custom-House, in Boston, Mass. The e vators and stairs are next to the west wall throughout the typical stories. Und this arrangement there is no adequate provision in the ordinary floor-constr tion for a wind-pressure on the north or south face to reach the resisting brac in the west face, as the various open wells cut off nearly all direct connecti between the floors and this wall. Flat plates were therefore added on top of t floor-beams at each floor-level, running out from the wall girders behind t wells into the main floor-construction, and attached at each end with conn tions sufficient to transmit the horizontal increment of the wind-pressure each floor to the bracing which resists it.

4. Arrangement of Wind-Bracing

Usual Position of the Bracing. As wind-pressure is assumed to be u formly distributed over the face of a building, it is best to arrange systems bracing, as nearly as may be, symmetrically about the axis of each face. I generally easier to conceal in the exterior walls the required knees, gussets, other braces, and bracing is usually placed there. When the lines of brac have been selected, the areas of wall-surfaces which bring wind-pressure to e are readily determined.

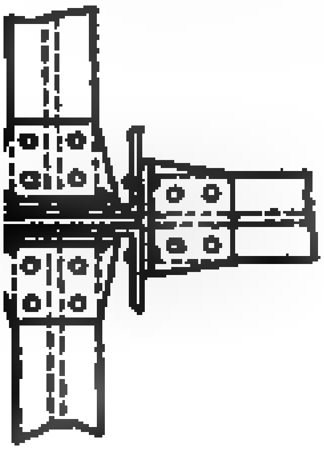
Bracing of Buildings of Irregular Plan. Some buildings are of such sh that it is impossible to provide bracing of equal stiffness in lines symmetri about the center of wind-pressure. This is notably true when the pla TRIANGULAR, as in the so-called Flatiron Building or in the Times Buildi New York City. The result is a tendency in such buildings to TWIST ABOU VERTICAL AXIS. The analysis of the resistance offered by a building to a t of this sort is unsatisfactory and complicated. The stresses produced in usual case, however, are small, if not negligible. In the examples mentio above, provision against twist is made by the use of deep spandrel girders around the building at each floor-level.

5. Types of Wind-Bracing

Ordinary Beam and Girder Column-Connections. Wind-bracing sh be so proportioned that the joints between horizontal and vertical members sufficient to prevent the distortion of the frame, and the main horizontal vertical members sufficient to resist any bending moments produced throug the joints, as well as any direct loads coming on them. The ORDINARY CON TIONS of steel beams to steel columns (Fig. 2) provide considerable resistan a distortion from side thrust. This is also true, of course, of connections bet beams and columns made of cast iron or concrete; but as these types are not adapted to construction where wind-bracing is required, they will not be sidered. A usual connection for beams or girders to columns consists of c ANGLES above and below the beam, and perhaps a STIFFENER below, if the is large. Usually, in high buildings, four rivets are used to connect either to the clip-angles above and below, and four to connect each clip to the col The value of four rivets, in single shear, multiplied by the depth of the b

TING VALUE of such a connection against a moment due to side
er buildings it is usual to specify two rivets instead of four in
l in the case of very high or narrow buildings, six rivets are some-

of Beam and Girder-Connections to Wind-Pressure. It is
med that the connections of all beams or girders (running in the



nary Girder and
onnections

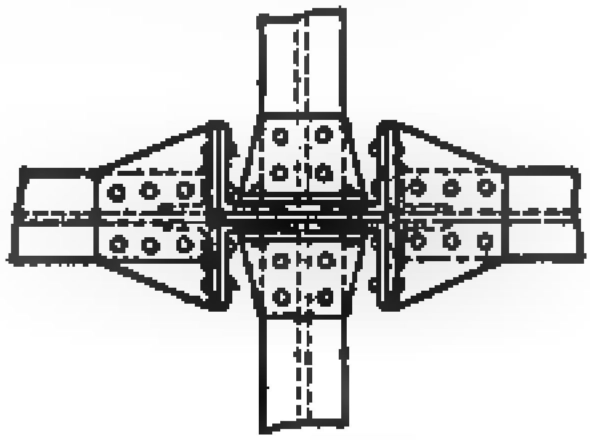


Fig. 3. Heavy Girder and
Column-connection

as the wind) to columns act at their full value to resist the wind-
otedly wrong, because the many connections could probably not
rk at the same time, and also because building-frames are seldom
at such a result could be possible, under any rational assumption,
he distribution of the vertical shears. SIDE CLIPS are sometimes
olumn-connections to furnish additional stiffness. They are not
, however, as on most beams they are not deep enough to help

umn-Connections in Wind-Resistance. Column-connections
made very heavy, as shown in Fig. 3. A connection of this kind
ed to resist a large TWIST. The RESISTING VALUE is, of course,
he resisting moment of the rivets connecting the beam to the clip
e connection of the angles to the column. This type is used where
to wind is provided for in a very large number of connections,
the column-connections, throughout the building. Such an

arrangement was used in the Hudson Terminal Buildings, New York City There are several objections to this type of connection. Double beams o girders are required, and the resulting finish is awkward in appearance; th cost, also, of double, compared with single, beams and girders, is high. Th additional fireproofing, also, increases the expense, and on the whole, it does no generally prove a satisfactory method of stiffening a building.

The Gusset-Plate Type of Wind-Bracing. In addition to ordinary bea and girder-connections, as described in the preceding paragraphs, there a several distinct types of special wind-bracing commonly employed. Perhap the most common form is the GUSSET-PLATE, shown in Fig. 4. This is no usually an economical type, as it requires much field-riveting and results i large bending moments in the columns and girders. It accommodates itse well, however, to walls in which there are openings, and is generally easily con cealed by architectural treatments.

The Knee-Brace Type of Wind-Bracing shown in Fig. 5 is also com monly used where wind-bracing is placed in exterior walls.

The Sway-Rod Type of Wind-Bracing shown in Fig. 6 is theoreticall the most economical type of wind-bracing, but is now little used. It is difficu to arrange openings in walls or partitions in which the sway-rods are place and they cut up the masonry considerably.

The Latticed-Girder Type of Wind-Bracing shown in Fig. 7 is sometim used where deep bracing is desirable for stiffness, and where the stresses a light.

The Portal Type of Wind-Bracing shown in Fig. 8 is cumbersome a expensive, but is sometimes necessary where large openings are required betwe columns.

6. Computation of Wind-Stresses

The Shears to be Transmitted by Wind-Bracing of any Type are t same in any given case, but the bracing of each type transmits these shears in different manner, and thus each must be considered separately. It is as thoug a PLATE GIRDER with SOLID WEB were set on end in the ground, a side thrust e erted, and the WEB THEN CUT AWAY at successive levels corresponding to t stories in a building. The amount of the shears to be transmitted between t flanges would not vary as holes in the web were made, but the road by which t shears traveled would need to be determined by the character of the resulti construction after the holes were cut; and the exact character of the seconda stresses, also, set up in the remaining portions of the web, would depend e tirely upon the number and size of holes and their position in the web. T investigation of the shears and moments taken care of by the individual membe of a bracing-system may be likened to a study of the secondary stresses in t mutilated web in the imaginary plate girder described above. For a buildi it is generally convenient to determine the VERTICAL-SHEAR INCREMENTS at ea level of bracing, and use these increments in the further analysis of the bendi moments and shears in the individual members of the system.

7. Illustration of Method of Computing Wind-Stresses

Thrusts, Vertical Shears and Moment-Increments. If bracing is plac in the walls *C* and *D*, Fig. 1, it is assumed that one-half the length of the build contributes pressure to each line. Let it be further assumed, for the prese that these lines of bracing are the only features of the construction offerin resistance to wind-bracing against side *A*. Then, assuming the wind to bl

:e per sq ft, perpendicular to side A, there are HORIZONTAL THRUSTS on each line of bracing, of 50 by 12 by 30 lb, or 18 000 lb. Re- e I, page 1178, there are found listed in the second column of the izontal thrusts at each floor. It is assumed that no additional reached the building below the fourth floor. In the third column se horizontal thrusts, ΣH, are summarized from the top down to giving the TOTAL HORIZONTAL THRUSTS. For example, 202 500 lb izontal thrust down to and including the tenth floor. Each tier t transmit a VERTICAL SHEAR equal to the difference in flange- a point midway in the story above the tier in question, and a 1 the story below. This difference of flange-stress can, of course, :ertaining the difference in bending moments between the two ding by the effective depth of the system, as in a plate girder or ow appear that the differences in moments applying to each tier ound and tabulated. These will be called the MOMENT-INCRE- ave been tabulated for the assumed case in the fifth column of ourse, the sum of all the moment-increments must equal the ING MOMENT of the wind. The simplest way to obtain the it for any tier is to multiply the total horizontal thrust, ΣH, el in question by the distance between points midway in the below the tier. Thus, for the tenth floor, 202 500 by 12 equals

nts of Vertical Shear are found by dividing the moment- e effective depth of the cantilever, in this case, 47 ft. The ENTS are listed in the sixth column of the table. It is full depth between outside columns as the effective depth of his is not strictly correct where there are four or more columns bracing, but the assumption is made on the ground that the nish flanges which are so many times more effective than the ins that the latter may be neglected. If there are a number lane of the bracing, say six or seven, this assumption becomes te, and the effective depth should be reduced. The function eretofore stated, is to carry between the flanges at each floor s of vertical shears thus found. The summation of all the from the top down gives the TOTAL VERTICAL LOAD and UP- on the corner-columns, or more correctly, on the outside r.

Shear. In this assumed case, the total uplift exceeds the live loads on the corner-columns. This, however, is not re are sufficient means furnished for transferring any excess the walls A and B, which act as flanges to the wind-resisting rule, the side walls of a city building are not much reduced higher buildings there are usually spandrel beams in the With such an arrangement a considerable amount of EXCESS are of. In some cases special bracing may be necessary, at s of walls A and B.

al or Flange-Stress. When considering any question AL LOAD OR UPLIFT, such as the one described in the pre- should be kept in mind that totals should be used without . Vertical forces forming couples to resist the wind must hey are transmitted through the masonry walls or through Referring again to the illustration of the plate girder set ertical shears are dependent only on the force of the wind

and the effective depth of the girder. The exact WEB-STRESS will vary with th form and arrangement of the web, but the TOTAL VERTICAL OF FLANGE-STRES must remain the same in any case.

Indeterminate Resistance-Factors. An analysis which makes no allowanc for the resistance of walls, ordinary connections, etc., to wind is fairly direc and simple, and the bracing can be proportioned with as much precision as an structural feature. When the wind-resistance of a building is a primary co sideration, as in a tower, the analysis should be made thus, for only in this wa can a result be obtained, where it is not required to rely on almost unsupporte judgment for the value of INDETERMINATE FACTORS OF RESISTANCE. Whe however, ordinary buildings of usual proportions are under consideration, it customary and well to make allowance for the INTEDERMINATE FACTORS, to th best of one's judgment. This is necessary for economy, and is perfectly prop so long as usual cases are to be dealt with.

Table I. Thrusts, Shears, Moment-Increments, etc., for the Building Shown in Fig. 1

I Floor	II Horizontal thrust at each floor, H	III Total horizontal thrusts from roof to each floor, ΣH	IV Arm, A	V Moment-increment, M	VI Vertical increment, V	VII Total vertical increments from roof down, ΣV	VIII Correct vertical increment,
	lb	lb	ft	ft-lb	lb	lb	lb
Roof	4 500	4 500	6	27 000	550	550	....
20	18 000	22 500	12	270 000	5 750	6 300	....
19	18 000	40 500	12	486 000	10 350	16 650	....
18	18 000	58 500	12	702 000	14 950	31 600	....
17	18 000	76 500	12	918 000	19 500	51 100	....
16	18 000	94 500	12	1 134 000	24 100	75 200	....
15	18 000	112 500	12	1 350 000	28 700	103 900	....
14	18 000	130 500	12	1 566 000	33 300	137 200	....
13	18 000	148 500	12	1 782 000	37 900	175 100	4 600
12	18 000	166 500	12	1 998 000	42 600	217 700	9 300
11	18 000	184 500	12	2 214 000	47 200	264 900	13 900
10	18 000	202 500	12	2 430 000	51 800	316 700	18 900
9	18 000	220 500	12	2 646 000	56 400	373 100	23 100
8	18 000	238 500	12	2 862 000	61 000	434 100	27 700
7	18 000	256 500	12	3 078 000	65 600	499 700	32 300
6	18 000	274 500	12	3 294 000	70 200	569 900	36 900
5	18 000	292 500	12	3 510 000	74 700	644 600	41 400
4	18 000	310 500	12	3 726 000	79 300	723 900	46 000
3	....	310 500	12	3 726 000	79 300	803 200	46 000
2	....	310 500	15	4 657 500	99 200	902 400	65 900
1	....	310 500	15	4 657 500	99 200	1 001 600	65 900

Scheme for Developing Special Bracing. The writer offers the follo as a reasonable and consistent scheme for DEVELOPING SPECIAL BRACING w such allowances are considered. Unfortunately, it does not seem possibl recommend any method of determining the correct allowances, except such g eral guides as are mentioned in Subdivision 2, page 1172. In each inst some one familiar with construction and usual practice should decide how

a the top it will be safe to assume the building rigid and secure against
iout special bracing. In this case, let it be assumed that the building
of safely resisting the wind, without the aid of special bracing, as far
ie thirteenth floor. Then, assuming that the walls, beam-connections,
in reasonably the same in the floors below, it is fair to say that the
f the increment at the fourteenth floor can be deducted from the
crement at each floor below.

ed Vertical Increments. The CORRECTED VERTICAL INCREMENTS
is manner should be used only in the proportioning of special bracing.
verturning moment of the wind, and the full vertical shears, should
considering all other effects of the wind and the resistance of the
it. It should also be borne in mind that this method of proportion-
; is, at best, largely dependent upon individual opinion, and in any
se it is far better to err on the safe side, even to the extent of disre-
ogether the uncertain factors of resistance. The CORRECTED VERTICAL
s for the assumed case have been listed in the eighth column of
Since the flanges of the building, acting as an upright cantilever
been assumed concentrated in the outside walls *A* and *B* (Fig. 1), it
t the VERTICAL-SHEAR INCREMENTS will be constant from outside to
racing.

alysis of Stresses in Different Types of Wind-Bracing

izontal Thrusts, which must be carried by the bracing at each level
re small and can usually be neglected. The MAXIMUM THRUST at
-level can
the hori-
ure of the
one story,
example,

**tal Hori-
ear.** The
the brac-
must carry
ORIZONTAL
ach story,
is usually
comparis-
shearing
the col-
n be neg-

Gusset-
4 repre-
al panel
BRACING.
luence of
re there

Fig. 4. Gusset-plate Type of Wind-bracing

Fig. 5. Knee-brace Type of Wind-bracing

dency to distort the frame, changing the angle between the vertical
members; that is, the columns and girders. For this investigation, a
wel is considered, with gusset-plates at either end, as shown. These
ct to prevent the distortion, and since they both, at either end, re-

sist the same wind-action, the twisting moment in each has the same sign. But they twist in the same direction at opposite ends of the girder, there is somewhe along the girder and between the gussets a POINT OF INFLECTION or a POINT OF N BENDING. The position of this point varies with the relative strength of the tw gussets, but for simplicity it is usually assumed midway between them and th are then proportioned to take care of the resulting moments. Let the point inflection in the example be thus taken. As there is no bending moment at th point, the bending moment at any other point on the girder may be found b multiplying V, the increment of vertical shear for the level in question, by t distance from the point of inflection. So, at the toe of the gusset-plate, t bending moment on the girder equals V multiplied by e, and this is the MAXIM BENDING MOMENT ON THE GIRDER. The flange-stress having been determin from this bending moment, it is possible to fix the number of rivets requir to fasten the flanges to the gusset. The connection of the web to the guss must provide for a shear equal to V. V multiplied by f gives the moment pr duced through the gusset at the face of the column. The rivets connecti the gusset to the column must be sufficient to resist this moment.

Points of Inflection occur in the columns midway between the gussets, ju as in the girders. The bending moment in the column may be obtained appro mately by assuming the moment exerted through the gusset-plates to be appl in the form of a couple acting at points two-thirds of the way out from the cen of the gusset to the tips, as indicated. The MAXIMUM BENDING MOMENT IN EA COLUMN will then be the horizontal force P multiplied by d. P is obtained multiplying V by l and dividing by c, l being the distance from the inflecti point in the girder to the axis of the column, and c being the distance betw the inflection-points in the column above and below the girder.

Gusset-Plates on Both Sides of Column. If there are GUSSET-PLATES TWO SIDES of a column, as is usual on interior columns, the maximum bendi moment in the column will be the sum of the maximum moments due to e gusset. Gusset-plate connections are easily arranged with plate girders or w double channels, or even with I beams.

Stresses in Knee-Braces. Let Fig. 5 represent a typical panel of KN BRACING. As described in the preceding paragraphs on gusset-plates, there m be POINTS OF INFLECTION, and consequently POINTS OF NO BENDING, in the gird and also in the columns. These points are assumed midway between the e of the knee-braces in both the columns and girders. Let V be the verti increment for the level under investigation.

Then $$P = Vl/c$$

and the reaction of the girder at the column,

$$R = Va/b$$

Since V and R act always in the same direction, S must be equal to their s Hence $$S = R + V$$

Maximum bending moment on girder $= Va$

or the equivalent

Maximum bending moment on girder $= Rb$
Maximum bending moment on column $= Pd$
Stress in each knee-brace $= \frac{1}{2} S$ cosecant α.

It is evident that R is the shear anywhere between the intersection-poi the center line of the knee-braces and the columns, and that V is the shear i

tween the braces at either end of the girder. All web-splices, and ›itch of flange-rivets, must be proportioned from these shears.

;ement of Braces for No Bending Moment in Girder or Column. ırent from the above that the nearer a and d approach zero the less the ıoments in the girder and column become. If the intersections of the es of the braces can be arranged so that a and d become zero, there) BENDING MOMENTS in the girders or columns.

3races on One Side, Only, of Girder. It is often necessary to :NEE-BRACES ON ONE SIDE, only, of the girder, either above or below. of this kind the girder itself serves as one arm of the brace, and

$$\text{The stress in the single knee} = S \text{ cosecant } \alpha$$

S are as determined above, but there must also be taken into account ntal stress in the girder, due to its action as one arm of the brace.

$$\text{Horizontal stress in girder} = Vl/(\tfrac{1}{2}c - d)$$

nection between the column and the girder must provide for the com- ›n of R vertical and $Vl/(\frac{1}{2}c - d)$ horizontal.

s in Sway-Rods. For the correct analysis of SWAY-BRACING (Fig. 6), l increments should be found in a manner slightly different from that ın Subdivision 1176–9. The pressures are before, except otal pressures top down to ıclude, in each lditional pres- st areas of one- y below. The ›r each level e the story- w. (See sec- 1 and fourth Table I.) The crements are s for the other pt for these ations; and, e vertical in- ıre constant each story. IN ANY DI- als the verti-

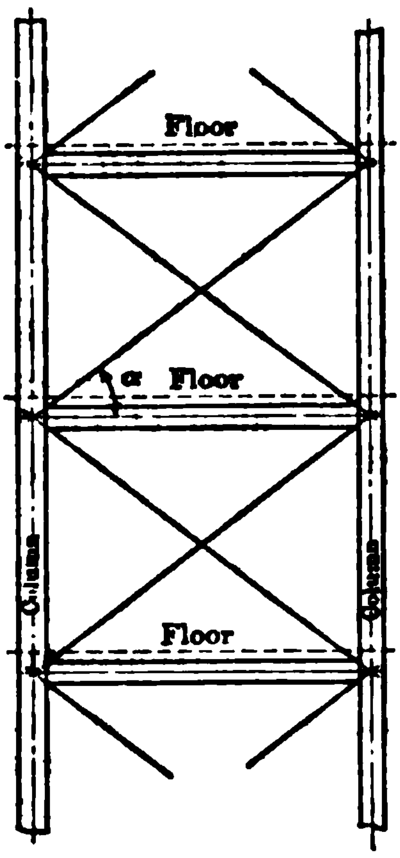


Fig. 6. Sway-rod Type of Wind-bracing

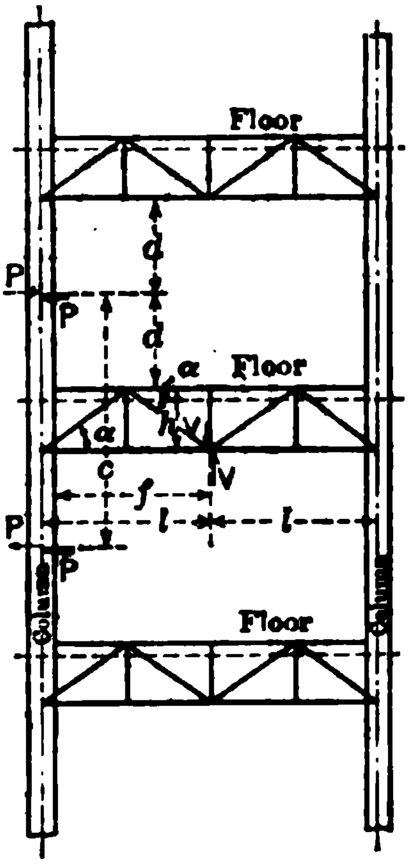


Fig. 7. Latticed-girder Type of Wind-bracing

ıt in the story multiplied by the cosecant of the angle α (Fig. 6). l that the diagonals are used for tension only and that, consequently, tem acts at a time. Each horizontal member must take compression vertical increment in the story below, multiplied by the cotangent e joints are arranged so that axial lines of members intersect, there ending either in the columns or the horizontal members.

in Latticed Girders. Let V in Fig. 7 equal the vertical increment ŗ. As in the other types, V is constant between the columns, and any diagonal equals V multiplied by the cosecant of α. As in the

GUSSET-TYPE and KNEE-BRACE TYPE, there is no bending at the middle secti of the girder-length, and consequently no stress in the middle section of t top chord. The maximum bending moment in the girder is at the column-fa and this

$$\text{Maximum bending moment in the girder} = Vf$$

The maximum chord-stress is at this same point, and this

$$\text{Maximum chord-stress} = Vf/h$$

The connections of the chords to the columns must provide against this m imum stress.

$$P = Vl/c$$

and the

$$\text{Maximum bending moment in the column} = Pd$$

Stresses in Portal Bracing. It is not possible to analyze exactly t stresses in PORTAL BRACING (Fig. 8), when it is used in connection with colum of continuous section. The analysis given follows that of C. T. Purdy in "Mod Framed Structures." It is considerably the safe side, and for ordinary cases well be followed. In a large building, wh much bracing of this type might be the exact form of the PORTALS should determined, and greater allowance made the effect of CONTINUOUS COLUMNS.

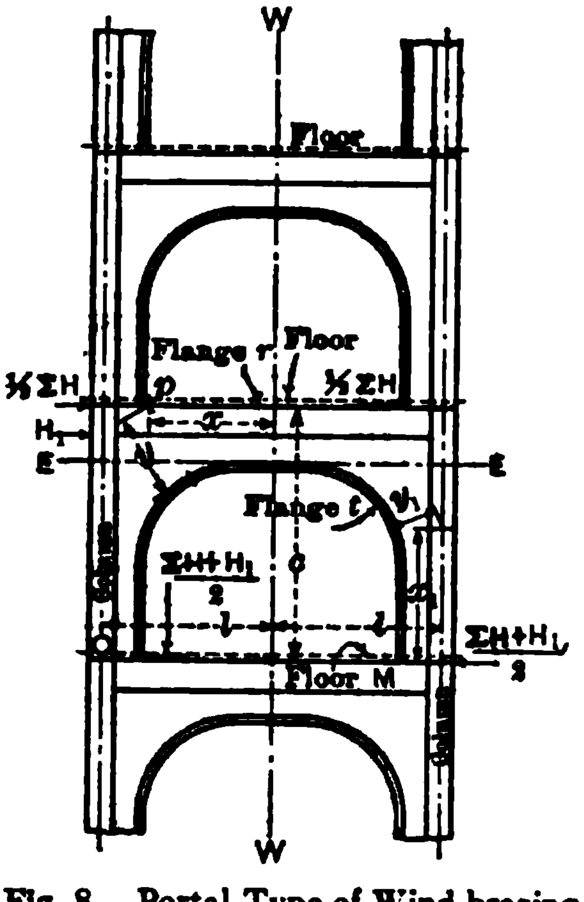


Fig. 8. Portal Type of Wind-bracing

Let ΣH equal the accumulated force horizontal shear from the wind at the fl next above floor M, applied half on side and half on the other. Let H_1 eq the force of the wind or the shear direc tributary to floor M. Then, taking mome about O (Fig. 8)

$$V \times 2l = (\Sigma H + H_1)c$$

or

$$V = (\Sigma H + H_1)c/2l$$

and the

$$\text{Horizontal reaction} = \tfrac{1}{2}(\Sigma H + H_1)$$

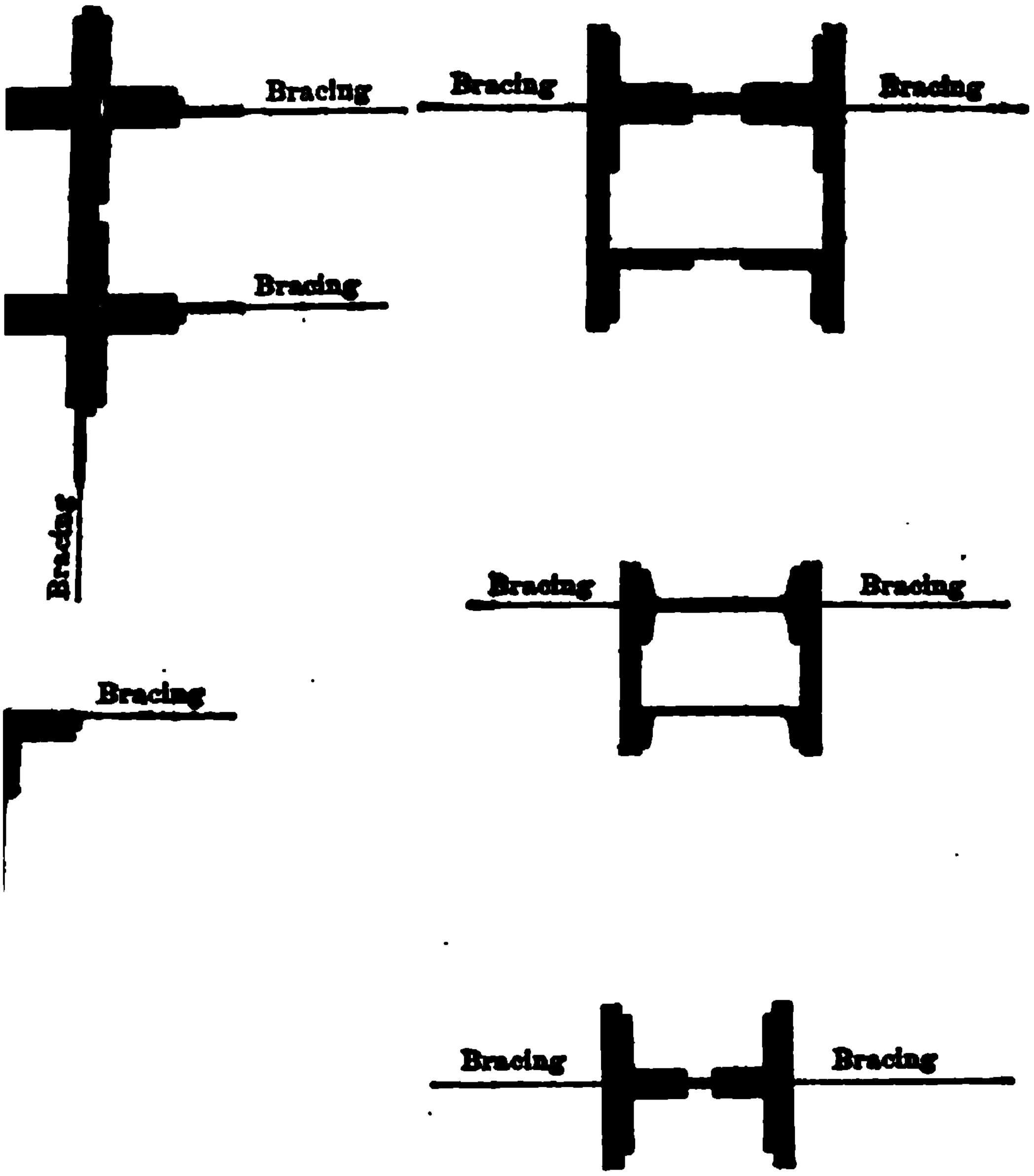
To determine the maximum stress in curved flange t, assume a point p in flange r, horizontally distant x from the line WW, and at the distance y, me ured normal to a tangent to any point in the flange t; then, taking center of moments at the left extremity of the distance x, the stress in flange t multiplied by y, equals V multiplied by x, or Vx/y equals the stres the flange t, at the section taken, and this is a maximum when x/y has its gre est value.

Each leg of the portal, including the column, may be considered as a CAN LEVER with two forces acting on it, the horizontal force $\tfrac{1}{2}(\Sigma H + H_1)$ and vertical force $(\Sigma H + H_1)c/2l$, the flange t (of the right leg for example) be in COMPRESSION and the column itself acting as a TENSION-CHORD. Assumin point on the axial line of the column, distant x_1 from the bottom of the leg at right-angles to, and distant y_1, measured normal to a tangent to any poin the flange t, and taking moments about this assumed point, the stress in flange t multiplied by y_1 equals $\tfrac{1}{2}(\Sigma H + H_1)$ multiplied by x_1, or the stres

uals ½ $(\Sigma H + H_1)$ multiplied by x_1/y_1, and this is a maximum its greatest value. There is approximation in this treatment, e side of safety. If the flange t has a section proportioned to stresses the requirements will be fulfilled. The stress in, and uired for, the flange r can be obtained in a similar manner. The e portal above this flange to the portal and column above must will safely resist the stress ½ ΣH at each leg.

ination of Dead and Live Loads with Wind-Loads

iciples. It usually happens that the same girders that are used serve also to carry floors or walls. The dead and live loads dered with the wind, and the RESULTANT COMBINED STRESSES



9. Types of Columns Arranged for Wind-bracing.

ould be borne in mind that the maximum bending moment l is often at a point on the girder more or less removed from um bending moment for dead and live loads. When RESULT-OMENTS are considered, in which the forces are the wind-load ead loads, it is generally deemed proper to use unit stresses hose of common practice under usual loading. The columns ted for direct live, dead and wind-loads and for the bending RESULTANT STRESS, again, should not exceed 150% of the sed for live and dead loads only. It is often best to design ecial view to proper connections for bracing. This aids in

both design and detail. In Fig. 9 are shown a few TYPICAL ARRANGEMENTS column-material illustrating this point.

10. Wind-Bracing of Water-Towers and Similar Structures

The Principles Involved in Water-Tower Bracing. In the case of TOWER WITHOUT MASONRY WALLS, a problem is presented much simpler th that of a building, as the INDETERMINATE FACTORS OF RESISTANCE are larg eliminated. The bracing sho be designed to resist the wind-pressure. It should borne in mind that in wat towers the condition of MINIM STABILITY obtains when the ta is empty. The most comm form for tower-bracing is SWAY-ROD. The analysis stresses is the same as describ on page 1181. The applicat of the thrust is largely at the where the tank stands, but t does not in any way alter the alysis. The legs of water-tow are frequently SLOPED to giv greater spread at the botto In this case the stresses are m readily determined by GRAP METHODS than by algebr or trigonometrical computati (See Chapter XXVII.)

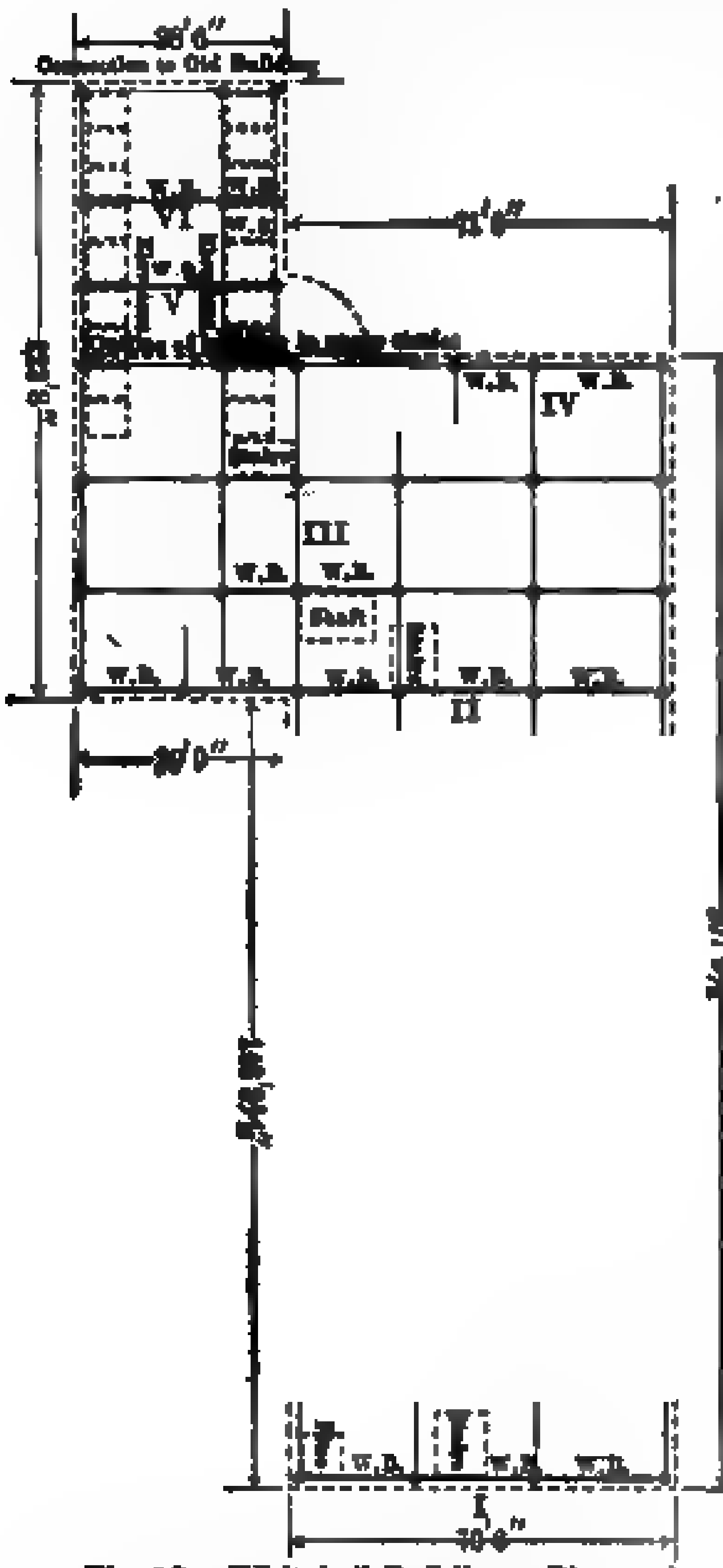


Fig. 10. Whitehall Building. Plan and Lines of Bracing

The Assumed Unit Pr sures should be somewh greater for towers than for bui ings. Towers are small in co parison with buildings, and t probability of the full wind-pr sure being developed over t entire surface is greater. Pr ably 40 lb per sq ft is amp Pressure against a CYLINDRIC BODY, such as a tank, may taken at about two-thirds of t full pressure against the proj tion on the diametrical pla The stresses under this assum pressure should be kept wit usual bounds for ordinary de or live loads. The anchorage each post should exceed, by a safe margin, the full uplift due to the assum pressure. The weight of water in the tank should not be considered as resisti the uplift.

A Good Example of a Steel Water-Tower is described and illustrated the Engineering Record of June 20, 1903, the stress-diagrams and details of co struction being given.

I

itehall Building. Wind-bracing on Line I, Fig. 10

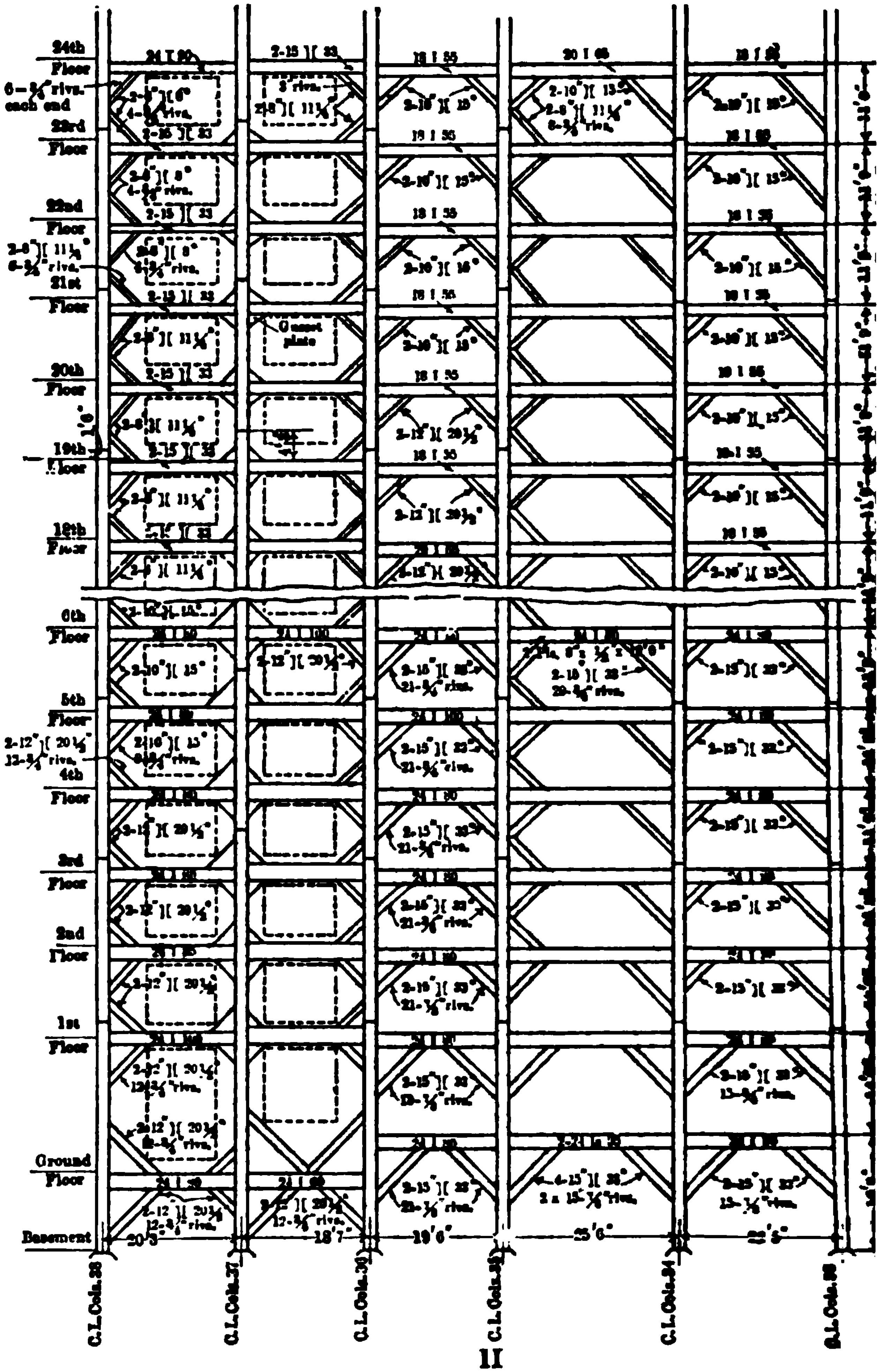


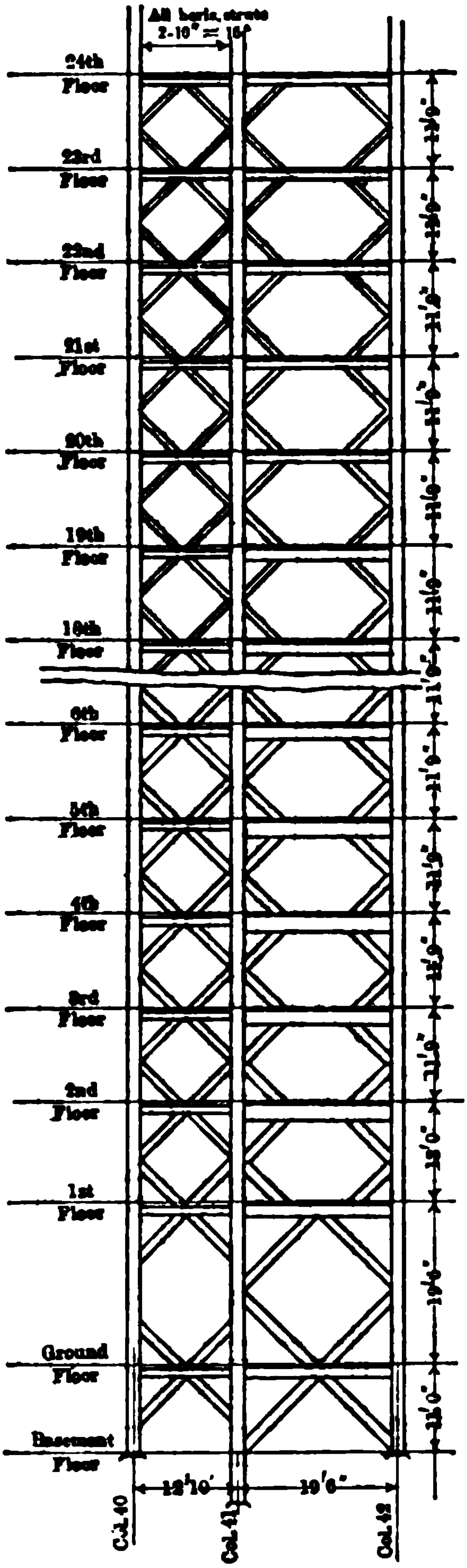
Fig. 12. Whitehall Building. Wind-bracing on Line II, Fig. 10

Recent Examples of Wind-Bracing in Tall Buildings *

hitehall Building† (Figs. 10 to 14 West Street, New York its of a thirty-one story addi- earlier Battery Place Building. : older twenty-story building he south. As the plan, Fig. 10, : building is very long and nar- mpared with its height and it ype in which wind-bracing must an essential feature. The six ng indicated on the plan (Fig. oman numerals and the letters osen so as to interfere as little h the requirements of the plan. were used as far as practi- everal instances it was neces- SSETS because of the limited e. It was assumed that the ections of girders to columns, furnished sufficient stiffness venty-fourth floor-level. Be- he bracing was proportioned pages 1179–80; that is, al- ade for the INDETERMINATE ISTANCE equal to the wind- twenty-fourth floor.

States Realty Building ‡ Broadway, New York City, ple of a building in which : is quite essential. It is ies high, and its width is pared with its length and was used, as indicated, in t it was not feasible to put lines to do all the work. were therefore added be- he elevator-shafts and in hown on the plan. No s used above the fifteenth

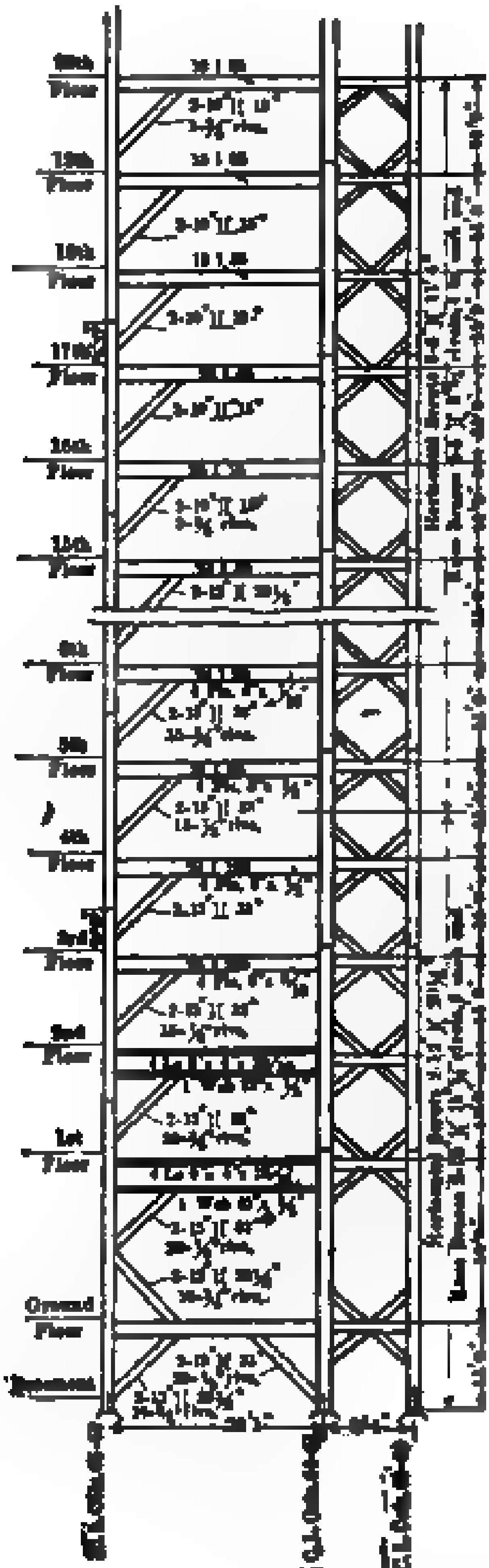
Building § (Fig. 19), 681 r York City, is but twelve is rather narrow. The interior lot, and it was the openings in the ex- : as possible, in order to nterior. This, of course, walls of but little value



III

Fig. 13. Whitehall Building. Wind-bracing on Line III, Fig. 10

[enderson acted as designing engineers for these buildings.
Russell, architects.
Kimball, architect.
Mead & White, architects.

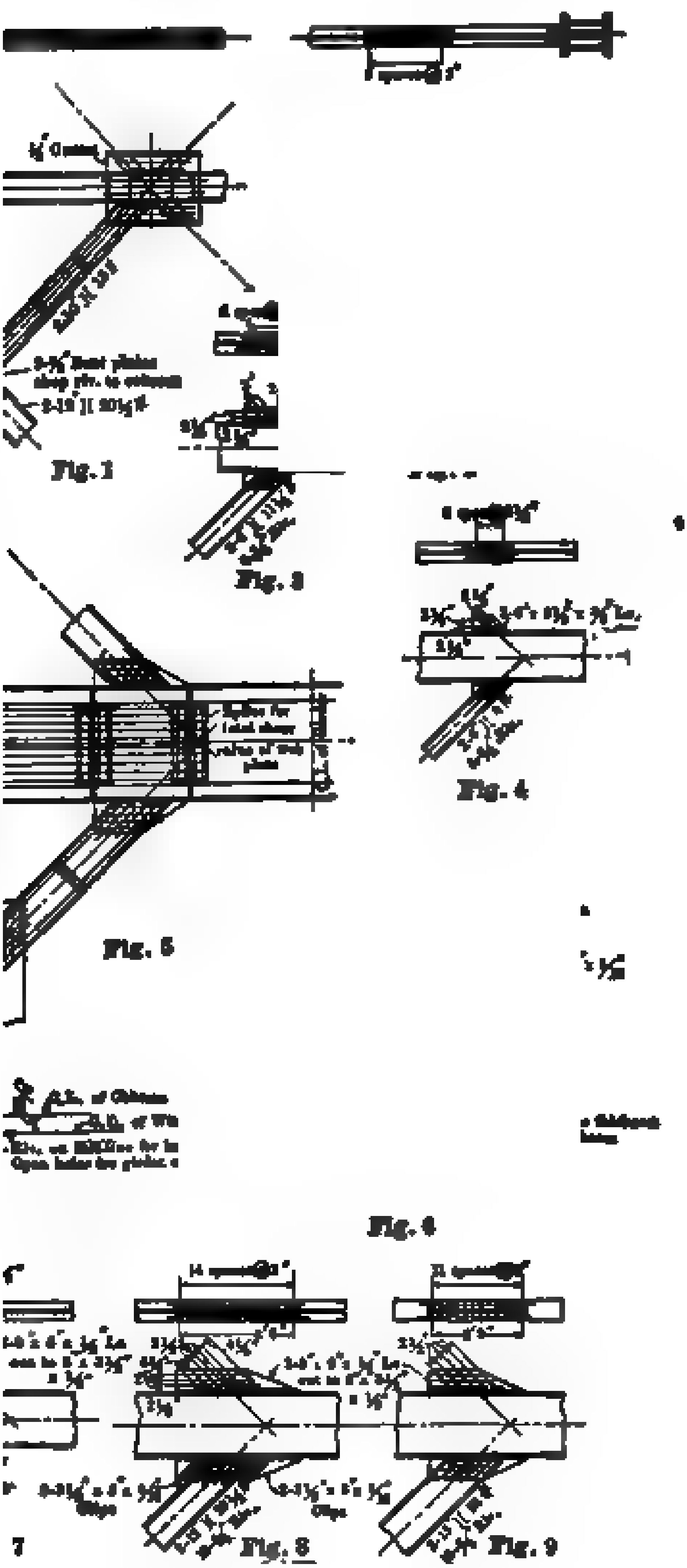


IV

Fig. 14. Whitehall Building. Wind-bracing on Line IV, Fig. 10

V and VI

Fig. 15. Whitehall Building. Wind-bracing on Line V and VI, Fig. 10



. 16. Whitehall Building. Wind-bracing Details

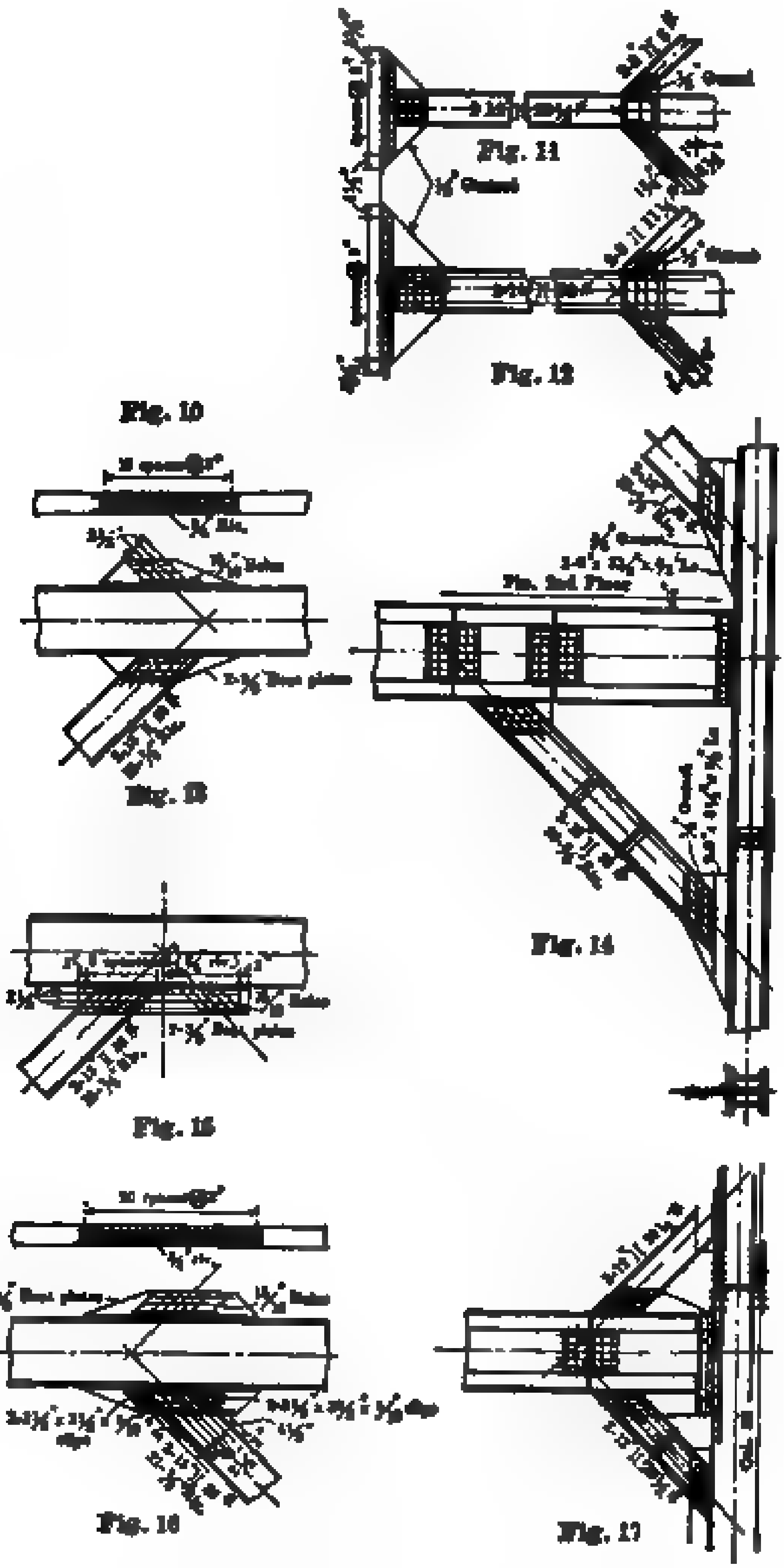
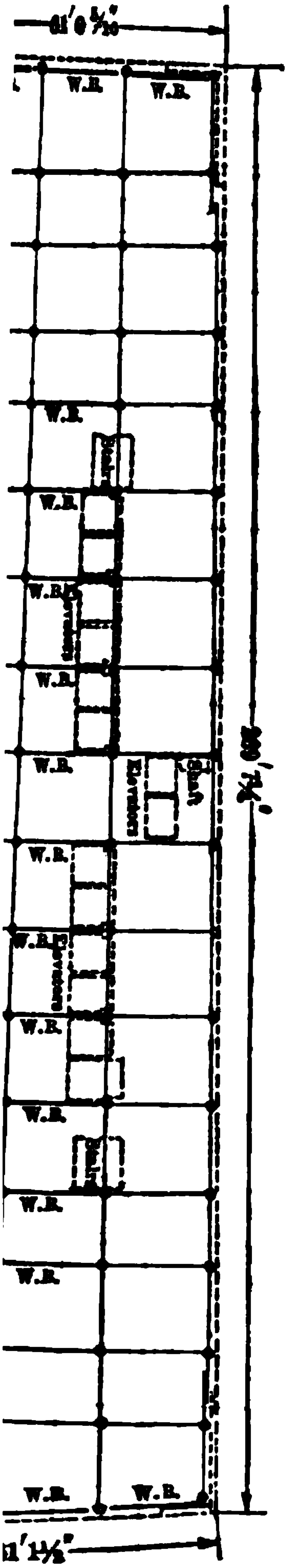


Fig. 17. Whitehall Building. Wind-bracing Details



ł States Realty Building.
Lines of Bracing

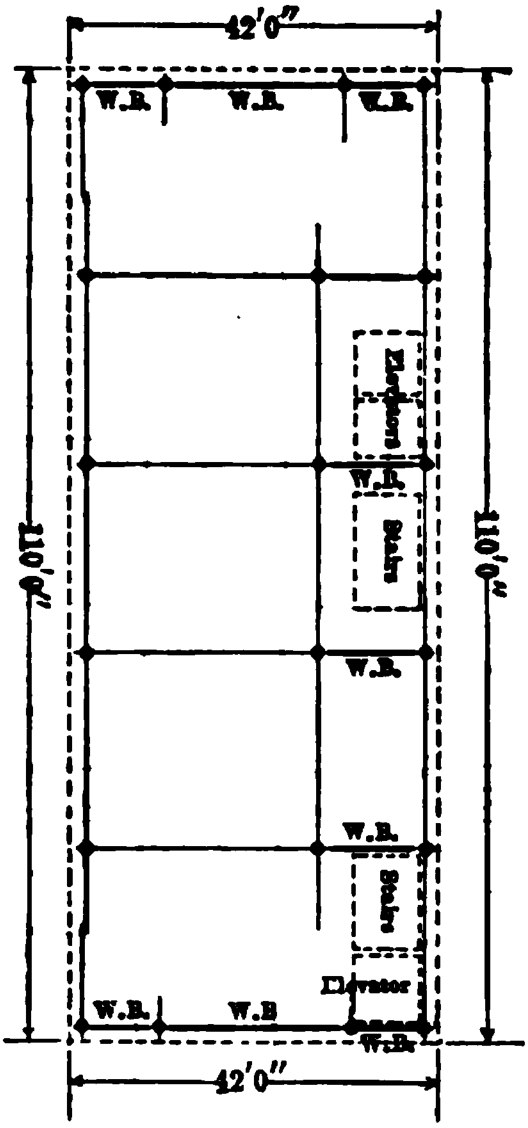


Fig. 19. Morton Building. Plan and Lines of Bracing

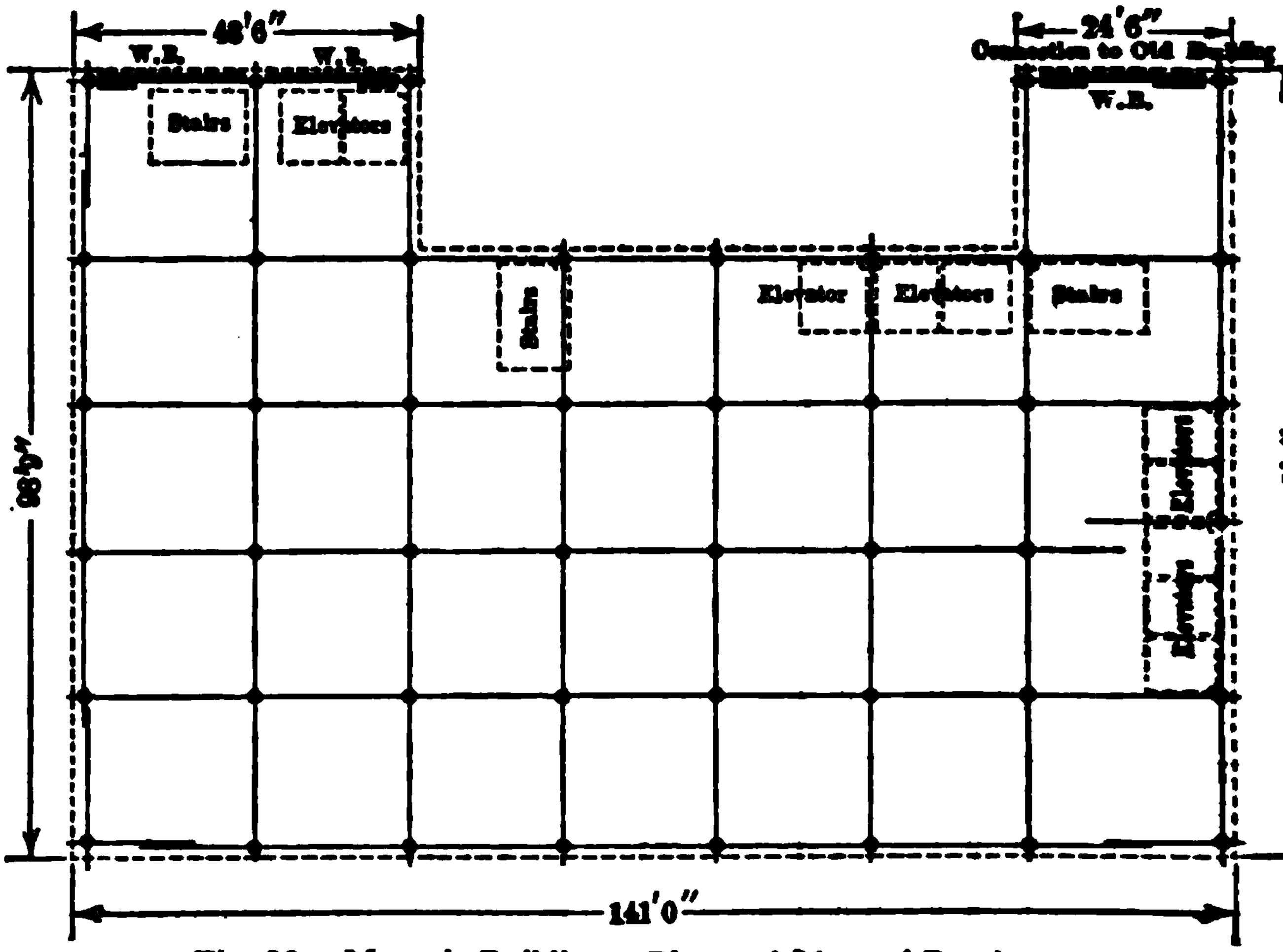


Fig. 20. Masonic Building. Plan and Lines of Bracing

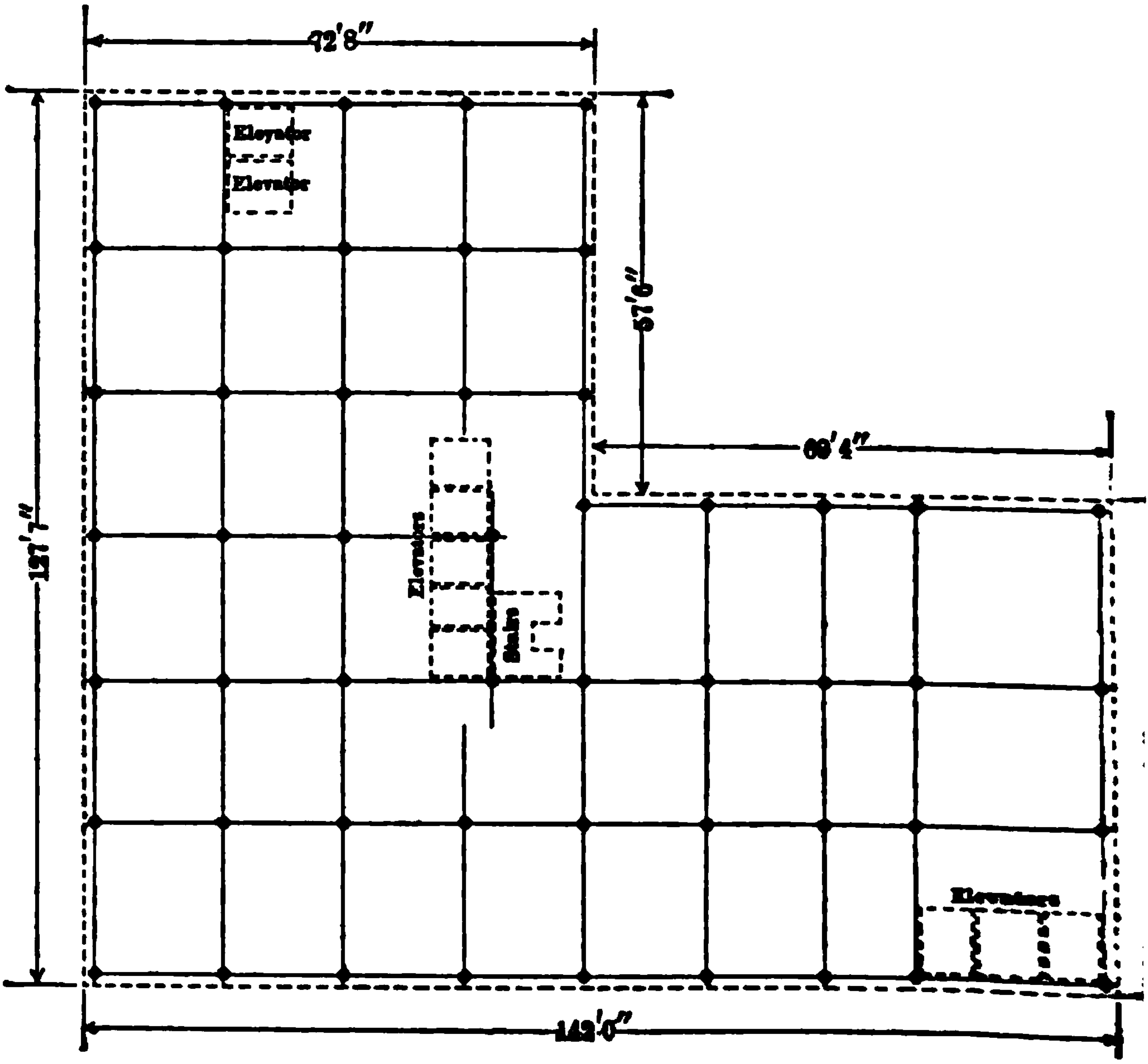


Fig. 21. Everett Building. Plan

ıg. Small KNEE-BRACES and GUSSETS were introduced in each end-additional knee-braces next to the elevator-wells and stair-wells. The ;irder-connections to the columns, also, were made with six rivets in ;e, instead of with four as is usual. These special bracing-features were) to the fifth floor-level.

'asonic Hall* (Fig. 20), 24th Street Building, New York City, twenty-es high, has virtually no special bracing. Light KNEE-BRACES were l in the two wings, but these were rather to insure the steelwork against ıt of plumb in erection, than to assist in wind-resistance.

'erett Building† (Fig. 21), 45 East 17th Street, New York City, is a ıry building, with no special bracing of any kind. It is large on the nd the ordinary features of construction offer ample resistance to

etropolitan Tower‡ (Fig. 22), Madison Square, New York City, is usual case that it is, perhaps, out of place to mention it as an example e typical. It is 700 ft bout 75 by 85 ft in plan ɔut the lower stories. ng in this case is a ıre of the structural de-ceived much attention. ıed from top to bottom ıll pressure of 30 lb per ı no reduction for the alls, etc. The bracing ı general, of PLATE the walls at each level, -BRACES and GUSSETS ıts. The columns are th especial view to the onnection for the brac-ad weight of the tower :ENT OF RESISTANCE to far in excess of the THE WIND.

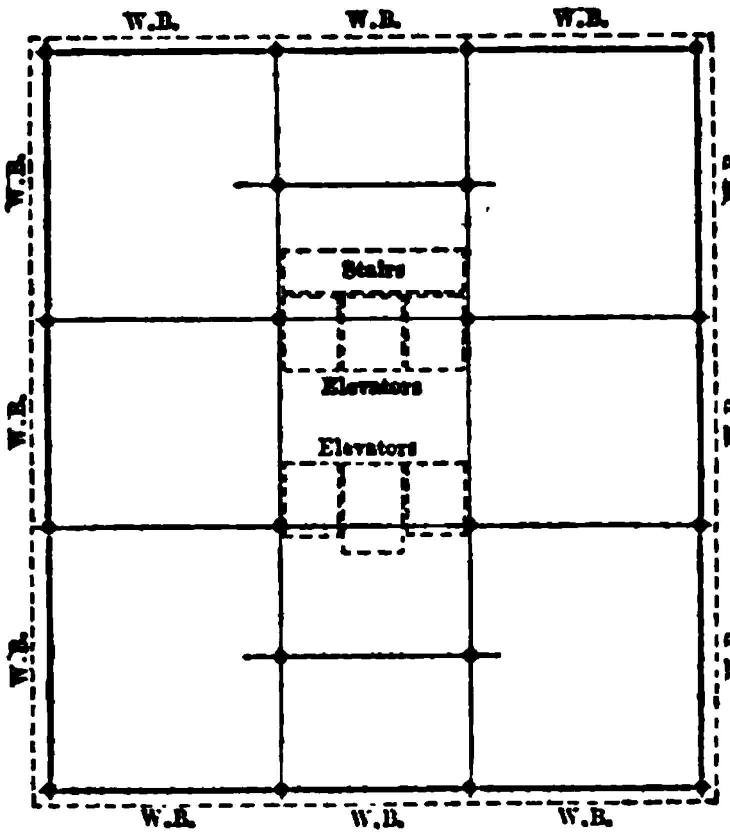


Fig. 22. Metropolitan Tower. Plan and Lines of Bracing

mples, all drawn from :ity buildings, are per-ical of the most ap-:rn practice in respect to wind-bracing as could be chosen. There is ite variety in the shape and size of buildings that no case is ever y previous example.

* H. P. Knowles, architect.
† Goldwin Starrett & Van Vleck, architects.
‡ N. LeBrun & Sons, architects.

CHAPTER XXX

SPECIFICATIONS * FOR THE STRUCTURAL STEELWO OF BUILDINGS. DATA ON STRUCTURAL STEEL

By

ROBINS FLEMING

OF THE AMERICAN BRIDGE COMPANY, NEW YORK, N. Y.

1. General

(1) **Drawings.** The drawings forming a part of these specifications [give number, maker, title, and date of each drawing].

(2) **Classification.** For the purpose of classification buildings are div into two classes:

I. Mill-Buildings

II. Office-Buildings

Under Class I are included manufacturing plants, machine-shops, po houses, rolling-mills, foundries, forge-shops, pattern and template-shops, t sheds, pier-sheds, car-barns, roundhouses, electric-light stations, armories, buildings of a similar character.

Under Class II are included office-buildings, hotels, apartment-houses, d ings, public buildings (hospitals, libraries, schools, court-houses, jails), plac public assembly (churches, theaters, halls), stores, warehouses, garages, buildings of a similar character.

(3) **Scope of Work.** It is intended that these specifications and dra cover the structural steelwork complete for the building. Cast-iron base included with the structural steel. The steel-erector shall erect in place steel framework on foundations furnished by others. Anchor-bolts, loose li and material not connected with main frame of structure, are to be delivere the site, but put in place by other contractors.

(4) **Materials to be Furnished for Buildings of Class I.** Unless spe otherwise in contract, the materials to be furnished for buildings of Cl include steel trusses, columns, purlins, bracing, floor-framing, crane-girders rails, trolley-beams, lintels, girts, framing around door and window-ope beams supporting tanks, elevator-framing, stair-framing, floor-plates, bu framing and steel lining, stairs and railings unless of an ornamental char cast-iron bases, grillage-beams, and anchor-bolts.

The materials not furnished include ornamental ironwork and steel masons' anchors, carpenters' anchors and irons, elevator sheave-beams, s for trolley-beams, steel stacks, steel tanks, and steel reinforcement for con

(5) **Materials to be Furnished for Buildings of Class II.** Unless spe otherwise in contract, the materials to be furnished for buildings of Cl include steel columns, cast-iron bases, rolled and cast-steel slabs, grillage-be anchor-bolts, floor-framing, roof and ceiling-framing, purlins, cornice-sup supports for tanks, penthouse framing, bracing, and lintels.

The materials not furnished include ornamental ironwork and stee masons' anchors, terra-cotta anchors, carpenters' anchors and irons, stair-

* The various values used in these Specifications agree generally with th throughout the book. Any slight variations in values are due to recognized and al differences in engineering judgment, differences in Building Codes, etc.

raming, elevator sheave-beams, steel stacks, steel tanks, light shapes g metal ceiling-lath, cast-iron sills and similar work, and steel reint for concrete.

VETS AND BOLTS for fastening steel to steel (but not for connecting the other trades) shall be furnished by the steel-contractor. Fitting-up erection are to be furnished by the contractor for erection as a part of nent.

soon as possible a COLUMN-FOOTING PLAN shall be sent to the purowing the location, elevation, and dimensions of all column-bases, with on, elevation, size, and length of all anchor-bolts. The loads coming column-footings from the columns shall also be given.

ine-clearance diagrams showing the CLEARANCES assumed for traveling ll be furnished the purchaser at an early date.

ostitution of Material. If the contractor wishes to substitute OTHER SIZES for those called for on the drawings he may do so, subject to the of the engineer, provided the architectural features are maintained and sections are sufficient to carry the required loads.

ork of Other Trades. HOLES conforming to the usual standards of shall be punched in the steel for attaching the work of other trades, heir location is given while the working drawings are being made.

orking Drawings. WORKING OR SHOP DRAWINGS shall be made by ontractor, and when requested, prints in duplicate sent to the purhis engineer for approval. The engineer's approval of drawings shall eral design, strength and type of details. The engineer shall not be nsible for the fit of work at the site. If, to expedite delivery, or for any on, he waives the approval of drawings, the contractor will not be f responsibility for errors or omissions due to neglect or oversight contractor's) part.

work shall conform to local or state ORDINANCES AND REGULATIONS.

2. Material *

operties and Tests. All parts of the metallic structure shall be of EEL, except column-bases, bearing plates, or minor details, which may IRON OT CAST STEEL.

uctural steel may be made by either the BESSEMER PROCESS or the TH PROCESS; except that rivet-steel, and steel for plates or angles over ickness which are to be punched, shall be made by the open-hearth

ictural steel, if made by the Bessemer process, shall contain not more ,; and if by the open-hearth process, not more than 0.06% of PHOSRivet-steel shall not contain more than 0.06% of PHOSPHORUS nor 0.045% of SULPHUR.

ictural steel shall have an ULTIMATE TENSILE STRENGTH of from 55 000 , and rivet-steel from 46 000 to 56 000 lb per sq in of cross-section. The IELD-POINT as determined by the drop of the beam of the testingall be one half of the ultimate tensile strength.

irements for material are taken, by permission, from the following Standard of the American Society for Testing Materials, Philadelphia, Pa.: Standard for Structural Steel for Buildings (A9-16), Standard Specifications for stings (A48-05), and Standard Specifications for Steel Castings (A27-16).

(17) The MINIMUM PERCENTAGE OF ELONGATION in 8 in shall be 1 400 0 divided by the ultimate tensile strength. For structural steel over ¾ in in thic ness, a deduction of 1 from the above percentage of elongation in 8 in shall made for each increase of ⅛ in in thickness above ¾ in, to a minimum of 18 For structural steel under 5⁄16 in in thickness, a deduction of 2.5 from the abo percentage of elongation shall be made for each decrease of 1⁄16 in in thickne below 5⁄16 in.

(18) TEST-SPECIMENS for plates, shapes, and bars shall bend cold through 18 without cracking on the outside of the bent portion, as follows: For materi ¾ in or under in thickness, flat on itself; for material over ¾ in, to and includi 1¼ in in thickness, around a pin the diameter of which is equal to the thickne of the specimen; and for material over 1¼ in in thickness, around a pin the dia eter of which is equal to twice the thickness of the specimen. The TEST-SPECIM for rivet-steel shall bend cold through 180°, flat on itself, without cracking the outside of the bent portion.

(19) IRON CASTINGS shall be of tough gray iron, true to pattern, free fro cracks, flaws, and excessive shrinkage. The SULPHUR-CONTENTS shall be n over 0.08% for light castings, 0.10% for medium castings, and 0.12% for hea castings.

A TEST-BAR 1¼ in in diameter and 15 in long, placed upon supports 12 in apa and tested under a centrally applied load, shall conform to the following requi ments:

MINIMUM APPLIED LOAD, 2 500 lb for light, 2 900 lb for medium, and 3 300 for heavy castings;

MINIMUM DEFLECTION AT CENTER, 0.10 in.

Light castings shall be able to withstand an ultimate tension of 18 000, medi castings 21 000, and heavy castings 24 000 lb per sq in. Castings having a section less than ½ in thick shall be known as LIGHT CASTINGS. Castings i which no section is less than 2 in thick shall be known as HEAVY CASTIN MEDIUM CASTINGS are those not included in the above classification.

(20) STEEL CASTINGS for ordinary use, not annealed, shall contain not mo than 0.30% of CARBON, nor more than 0.06% of PHOSPHORUS. They shall su stantially conform to the sizes and shapes of the patterns, be made in a wor manlike manner, and be free from injurious defects.

3. Loads

(21) **Roof-Loads.** ROOF-TRUSSES AND COLUMNS shall be designed to car a uniform load per square foot of exposed roof-surface, applied vertically. Th load includes the weight of the structure, the snow, and the wind. For spans u to and including 90 ft, and in climates corresponding to that of New York, t total minimum uniform load in pounds per square foot of roof-surface for differe kinds of covering shall be taken as follows:

	lb p sq
Corrugated metal	[illegible]
Gravel or composition on wood sheathing	[illegible]
Slate on boards	[illegible]
Tile on steel purlins	[illegible]
Gravel or composition on cinder concrete	[illegible]
Gravel or composition on stone concrete	[illegible]
Slate or tile on cinder concrete	[illegible]
Slate or tile on stone concrete	[illegible]
Fire-proof buildings of CLASS II, where slope is less than 2 in per ft	[illegible]

r roof-spans over 90 ft, the above-cited loads shall be increased 1% 2-ft increase of span.

r roofs in climates where snow is excessive, from 5 to 10 lb per sq ft lded, and in climates where there is not liable to be snow, 10 lb per sq ft educted from the foregoing loads.

a ceiling is carried by the roof-framing, the ceiling-load shall be assumed than 10 lb per sq ft.

SHAFTING is carried by the bottom chord, the load at the shaft shall be t not less than 2 000 lb for light shafting, 4 000 lb for ordinary shafting, lb for heavy shafting. Unless the shafting is definitely located these be considered as liable to be concentrated at any point of the bottom

designing PURLINS carrying roof-covering only, the loads in Para-) and (23) may be decreased 5 lb and considered normal to the roof. pitch of the roof is more than from 2½ in to 1 ft, tie-rods shall be used e purlins.

CIAL LOADINGS, such as tanks or elevator-supports above the roof, or trolleys on the bottom chord, shall be taken into consideration.

AT ROOFS used as places of public assembly or for storage-purposes nsidered as floors.

or Loads. Floor loads consist of DEAD LOADS and LIVE LOADS. The s composed of the weight of the floor-construction and of any perma-esting upon it. In designing floor-beams and girders for fire-proof n the dead load shall be assumed at not less than 70 lb per sq ft. of wooden studding or of hollow tile, not more than 4 in thick, may be as part of the live load.

ess governed by a local or state building code, buildings of CLASS I signed for MINIMUM LIVE LOADS, in pounds per square foot of floor-ows:

	lb per sq ft
pattern and template-shops	60
ops, light machinery	120
ops, heavy machinery	150 to 200
	150 to 200
charging-floor	300 to 800
es	200

LDINGS FOR SPECIAL INDUSTRIES shall be designed for the loadings those industries.

vision shall be made for the SUPPORT OF MACHINERY, engines, boilers, other concentrated loads, when carried by the steel construction.

ne-Loads. Loads due to electric TRAVELING CRANES shall be in-, to provide for the effects of impact. For hand-power cranes the be taken at 10%. For two cranes in action on the same girder, no be added, provided the stress obtained is larger than the stress due rane with impact. In addition to the vertical loads the top flanges ers shall be designed to resist a transverse horizontal thrust on each plied at the wheels, of 10% of the lifting capacity of the crane. The ce due to stopping the crane shall be assumed at 20% of each wheel-y be considered as distributing itself along the entire length of the

(34) **Coal-Bunkers.** COAL-BUNKERS shall be assumed to be surcha when it is possible for them to be so loaded. The weight of anthracite c shall be taken at not less than 50 lb per cu ft, and the angle of repose assumed be 30°.

(35) Buildings of CLASS II shall be designed for minimum live loads, in pou per square foot of floor-area as follows:

	lb sq
Dwellings (private residences), first floor	
Dwellings (private residences), upper floors	
Apartment-houses, first floor	
Apartment-houses, upper floors	
Hotels, first floor	
Hotels, upper floors	
Office-buildings, first floor	1
Office-buildings, upper floors	
School-buildings, class-rooms	
School-buildings, assembly-rooms	
Churches and theaters	
Places of public assembly, where floors are used for drilling or dancing	1
Places of public assembly, where floors are not used for drilling or dancing	1
Retail stores, ordinary	1
Warehouses	200 to
Private garages, pleasure vehicles only	
Public garages, pleasure vehicles only	
Garages, motor trucks, from 1 to 3 tons capacity	1
Garages, motor trucks, from 3½ to 5 tons capacity	

(36) CONCENTRATED LOADS shall be taken into consideration. Every st beam in any floor used for business purposes shall be capable of sustaining a l load, concentrated at the middle, of not less than 3 000 lb. Every steel beam the floor of a garage shall be capable of sustaining a concentrated live load 2 000 lb, if a private garage storing pleasure vehicles only; of 3 000 lb, if a pub garage storing pleasure vehicles only; of 8 000 lb, if motor trucks of from 1 to tons capacity are stored; and of 12 000 lb, if motor trucks of from 3½ to 5 to capacity are stored. Structural members carrying elevators and elevat machinery shall be proportioned to carry twice the actual moving dead and l loads.

(37) **Reduction of Live Load.** The full live FLOOR LOAD shall be used in p portioning all parts of buildings designed for warehouses, and such buildings are likely to be loaded on all floors at the same time. In other buildings specified live load may be reduced 10% for girders carrying 200 sq ft or more floor. For COLUMNS the load on the top floor may be assumed at 90% of specified live load; the live load on the floor next below the top floor at 8 and on each succeeding lower floor at correspondingly decreasing percenta provided that on no floor shall less than 50% of the specified live load be u and that for the lower floor the full specified live load shall be used. No red tion shall be made for any ROOF LOAD.

(38) In calculating COLUMN LOADS no reduction of floor-area shall be made stair-wells. Stairways shall be proportioned for not less than 75 lb per sq f horizontal projection.

(39) **Wind-Pressure.** Wind shall be assumed blowing horizontally in a direction. The surface exposed to WIND-PRESSURE shall be measured vertic from the ground to the top of the structure, including the roof.

hen the OVERTURNING MOMENT due to wind-pressure exceeds 75% of
nt of stability the structure shall be securely anchored.

ll steel buildings belonging to CLASS I shall be designed to carry wind-
o the ground by steel framework. For buildings not more than 25 ft
e-line the wind-pressure shall be assumed at not less than 15 lb per sq
corresponding normal pressure on the roof. For buildings more than
eave-line the wind-pressure shall be assumed at not less than 15 lb per
e lower 25 ft and 20 lb per sq ft for the side surface above 25 ft, and
ponding normal pressure on the roof.

e steel framework of fire-proof buildings belonging to CLASS II, in
height is more than twice the minimum horizontal dimension, shall be
o resist a wind-pressure of not less than 20 lb per sq ft on the sides,
rresponding normal pressure on the roof.

e normal pressure, P_n, in pounds per square foot, on a surface inclined θ
the horizontal for a horizontal wind-pressure, P, of 20 lb per sq ft,
to the DUCHEMIN FORMULA,*

$$P_n = P \frac{2 \sin \theta}{1 + \sin^2 \theta}$$

s:

	P_n lb per sq ft	Slope	θ	P_n lb per sq ft
	3.46	1 in to 1 ft	4° 45′ 49″	3.30
	6.76	2 in to 1 ft	9° 27′ 45″	6.39
	9.63	3 in to 1 ft	14° 2′ 10″	9.14
	12.25	4 in to 1 ft	18° 26′ 6″	11.50
	14.35	5 in to 1 ft	22° 37′ 12″	13.42
	16.00	6 in to 1 ft	26° 33′ 54″	14.88
	17.28	7 in to 1 ft	30° 15′ 24″	16.06
	18.20	8 in to 1 ft	33° 41′ 25″	16.95
	18.88	....	....	....
0°	20.00	....	....	....

ssure other than 20 lb per sq ft these values are to be changed pro-
.

excess of the wind-stresses, determined by the data of this paragraph,
nd-stresses according to Paragraph (21), need be considered. In
this excess, the wind included in the total uniform roof-loads desig-
agraph (21), shall be assumed at 5 lb per sq ft for slopes of 3 in per ft
l 10 lb for slopes more than 3 in per ft.

ULAR STEEL CHIMNEYS and TANKS shall be designed to resist a wind-
ot less than 20 lb per sq ft on the projected area, that is, the diam-
ed by the height.

SIGNS on tops of buildings shall be designed to withstand a wind-
ot less than 30 lb per sq ft of surface.

4. Stresses †

king Stresses. In proportioning structural steel for stresses due
TED DEAD AND LIVE LOADS together with IMPACT, the working stresses

page 1053.
variations from these values, see Table XVIII, page 618, and Table I,

in pounds per square inch of sectional area shall be not more than the fo
lowing:

	lb pe sq i
Tension, net section, rolled steel	16 0
Direct compression, rolled steel and steel castings	16 0
Bending on extreme fibers of rolled shapes, built sections, girders, and steel castings, net section	16 0
Bending, on extreme fibers of pins	24 0
Shear, on shop-rivets and pins	12 0
Shear, on field-rivets	10 0
Shear, on bolts	9 0
Shear, average, on webs of plate girders and rolled beams, gross section	10 0
Bearing pressure, on shop-rivets and pins	24 0
Bearing, on field-rivets	20 0
Bearing, on bolts	18 0
Tension, in rivets	7 0
Tension, in field-bolts (not anchor-bolts)	9 0
Axial compression, on gross section of columns and struts	16 000 − 70
where l is the effective length of the member, in inches, and r is the least radius of gyration of section, in inches, with a maximum of	13 0

(47) For COMBINED STRESSES due to wind and other loads the unit stresses
Paragraph (46) may be increased 50%, except for Paragraph (44), provid
the section thus obtained is not less than that required if wind-forces a
neglected.

(48) When the laterally unsupported length, l, of the compression-flange
beams and girders exceeds 12 times its width, b, the UNIT STRESS IN THE CO
PRESSION-FLANGE, shall not exceed $19\,000 - 250l/b$.

(49) COUNTERSUNK RIVETS in plates of thickness equal to or greater th
one half the diameter of rivet shall be assumed to have three fourths the value
rivets with full heads. In plates of thickness less than one half the diamet
of rivet their values shall be taken as three eighths that of full-headed rive
RIVETS WITH FLATTENED HEADS of height not less than ⅜ in, or one half the dia
eter of the rivet for ⅝-in rivets and less, may be assumed to have the value
corresponding rivets with full heads. Rivets with heads flattened to less th
these heights shall have countersunk holes and be regarded as countersu
rivets.

(50) The allowable pressure of COLUMN-BASES and BEARING-PLATES
masonry shall not exceed, in pounds per square inch, the following. (See, al
pages 265 to 267, and 441.)

	lb p sq
On brickwork, cement mortar	4
On brickwork, lime mortar	1
On Portland-cement concrete, 1 : 2 : 4 mixture	5
On Portland-cement concrete, 1 : 3 : 5 mixture	3
On rubble masonry, cement mortar	4
On rubble masonry, lime mortar	1
On first-class dimension sandstone	4
On first-class limestone	5
On first-class granite	6

5. Design

ieneral Design-Requirements. TRUSSES shall be riveted structures. -MEMBERS as well as COMPRESSION-MEMBERS shall be composed of rolled · built-up sections. Flat bars with riveted ends shall not be used.

n calculating TENSION-MEMBERS, net sections shall be used. The diam- ivet-holes shall be assumed to be ⅛ in larger than the nominal size of In single angles connected by one leg, the net area of the connected ne half that of the outstanding leg shall be considered effective.

he NOMINAL SIZES OF RIVETS shall be used in calculations of their values.

ı proportioning COLUMNS provision shall be made for eccentric loading.

OLUMNS AND STRUTS with direct loads of 40 000 lb or less, when ıall have the entire load transmitted through splice-plates.

OLUMN-SPLICES shall be designed to resist the bending-stresses, and to columns practically continuous for their whole length.

embers subject to REVERSAL OF STRESS from moving loads shall be ıed for the stress requiring the larger section, but their connections roportioned for the larger stress plus one half the smaller.

IE EFFECTIVE LENGTH OF MAIN COMPRESSION-MEMBERS shall not exceed their least radius of gyration, and for secondary members and lateral 60 times their least radius of gyration. Any portion of the cross- a compression-member may be neglected in computing the radius of provided that portion is neglected in the design of the member.

HEEL-LOADS OF CRANES shall be assumed to be distributed on the top runway girders over a distance equal to the depth of the girder, with a of 30 in.

ATE GIRDERS shall be proportioned either by the moment of inertia of ection, or upon the assumption that the bending-stresses are resisted nges concentrated at their centers of gravity, and that the shear is ' the web. When the second method is used one eighth of the gross the web, if properly spliced, may be used as flange-section.

EB-PLATES OF GIRDERS shall have a thickness of not less than 1/160 of ported distance between flange-angles.

ANGE-PLATES OF GIRDERS shall be limited in width, so as to extend han 6 in beyond the outer line of rivets connecting them to the angles.

EB-STIFFENERS, in pairs, shall be placed over bearings, at points of ed loadings and at intermediate points, usually not farther apart than th of the girder, when the thickness of the web is less than 1/60 of the d distance between flange-angles.

FFENERS under concentrated loads and over bearings shall be designed , with a length equal to one-half the depth of the girder, and shall h rivets to properly transmit the shear. When loads are transmitted e bearing of stiffeners, the bearing value may be assumed at 24 000 ı of section, excluding the area of the chamfered portion over fillets gles.

DEPTH OF GIRDERS AND ROLLED BEAMS in floors shall be not less than span, and if used as roof-purlins shall be not less than 1/32 of the span. loors subject to shocks and vibrations the depth shall be limited to pan.

EL PURLINS shall be single rolled shapes, plate girders or lattice

(67) Lateral, longitudinal, and transverse bracing in all structures sh preferably be composed of rigid members, and shall be designed to withst wind and other lateral forces when building is in process of erection as well after erection.

(68) Wind-bracing shall be provided for tall buildings by making the co nection-joint between girders and columns sufficient for the bending due to si pressure as well as for the vertical load; or diagonal bracing shall be plac between columns, proportioned to transfer the shear of the side pressure to t footings.

(69) No steel in any structural member subject to stress shall be less than in thick, except the webs of rolled beams and channels. Steel subject to t action of harmful gases or severe atmospheric conditions shall be not less th $\frac{5}{16}$ in thick.

6. Details

(70) **General Detail Requirements.** Details throughout shall confor to first-class standard practice.

(71) No connection except lattice-bars shall have less than two rivets, prefe ably three, for better handling in fabrication.

(72) In cases where it is necessary to carry loads subject to shock by bolts tension, check-nuts shall be used. When bolts go through beveled flang beveled washers to match shall be used so that head and nut are parallel. I general, rivets and bolts in tension shall be avoided as far as practicable.

(73) Abutting Joints in compression-members faced for bearing shall spliced sufficiently to hold the connecting members accurately in place.

(74) When two or more rolled beams are used to form a girder, they shall connected by bolts and separators at intervals of not more than 6 ft. A beams having a depth of 12 in and more shall have at least two bolts to ea separator.

(75) The minimum distance between centers of rivet-holes shall be thr diameters of the rivet, and the maximum distance in the line of stress eight dia eters.

(76) The minimum distance from the center of any rivet-hole to sheared edge shall be $1\frac{1}{2}$ in for $\frac{7}{8}$-in rivets, $1\frac{1}{4}$ in for $\frac{3}{4}$-in rivets, $1\frac{1}{8}$ in f $\frac{5}{8}$-in rivets, and 1 in for $\frac{1}{2}$-in rivets; and to a rolled edge, $1\frac{1}{4}$, $1\frac{1}{8}$, 1, and $\frac{7}{8}$ i respectively.

(77) The maximum distance from the center of any rivet-hole to an edge shall be eight times the thickness of the plate.

(78) The pitch of rivets at the ends of built compression-members sh not exceed four diameters of the rivets for a length equal to one-and-one-ha times the maximum width of the member.

(79) The latticing of compression-members shall be proportioned to resi a shearing-stress equal to 2% of the direct stress. Tie-plates shall be provid at each end and at intermediate points where latticing is interrupted. In ma members carrying calculated stresses, the end tie-plates shall have a length n less than the distance between the lines of rivets connecting them to the flang and intermediate ones not less than half this distance. Their thickness shall not less than $\frac{1}{50}$ of the same distance.

7. Workmanship

(80) **General Requirements.** All workmanship shall be first-class in eve respect.

laterial shall be thoroughly STRAIGHTENED before being worked, by that will not injure it.

SHEARING shall be done accurately, and all portions of the work exposed ieatly finished.

BUTTING SURFACES OF COMPRESSION-MEMBERS, except where joints are ced, shall be planed to an even bearing so as to give close contact it.

UNCHING shall be done accurately, but occasional inaccuracies in matches may be corrected with a reamer. The diameter of the punch shall re than 1/16 in larger, nor that of the die 1/8 in larger than the diameter et. Rivets shall be driven by pressure-tools wherever possible.

OLES IN MATERIAL of same thickness as diameter of punch may be ull size.

EB-STIFFENERS OF PLATE GIRDERS under concentrated loads shall have nilled.

8. Painting

neral Painting Requirements. Cast iron need not be painted at Steelwork for foundations to be entirely embedded in concrete shall ited, but must be free of dirt, grease, or other matter which would impair f the concrete. Other steelwork shall be thoroughly cleaned and given f paint before shipment. One coat shall be given to surfaces that are e after being riveted together.

ichine-finished surfaces shall be coated with white lead and tallow ment.

er erection all structural metalwork shall be cleaned of dirt and rust ne coat of paint of a color or shade different from that of the shop-coat.

painting at the shop and site shall be done by hand when the surface l is perfectly dry. Painting shall not be done in freezing weather.

nt shall be a good quality of red lead or graphite, ground in pure lin- their equivalent.

9. Inspection

neral Requirements. All inspection and tests shall be made at the expense of the purchaser.

iaterial is tested at the mills, the necessary number of test-pieces and testing-machine shall be furnished free of charge by the steel-con-

purchaser or his representative shall have free access at all times where material is rolled and to the shops where it is fabricated. In for his needs he shall be given dates of mill and shop-operations ed with complete working drawings.

10. Erection

eral Requirements. The structural steel and iron, except anchor- intels, and material not connected with the main frame of the struc- e erected by the steel-contractor on foundations furnished by the

shall be taken that all steelwork is level and plumb before bolting or

(97) Proper provision shall be made for resisting stresses due to erection operations.

(98) In general, field-connections shall be riveted, but connections of the following classes may be bolted:

(*a*) Light subordinate framing, such as purlins, monitor and skylight-framin girts, platforms, stair-framing, partitions, ceilings, and penthouses;

(*b*) Ordinary framing of beams to beams, and beams to girders;

(*c*) Connections not subject to direct shearing-stress.

All connections, however, affected by loads that cause undue vibration, sh be riveted. One-story buildings, not subjected to excessive wind-pressure not supporting heavy concentrated loads, shafting, or moving loads, may bolted throughout. The threaded part of a bolt shall not be so long that t bearing value of the unthreaded portion is reduced to less than the sheari value of the bolt. Washers shall be used under nuts wherever needed.

(99) Drift-pins shall be used only to bring parts together. Unfair holes sh be made to match by reaming.

(100) After finishing the work the erector shall remove his equipment and rubbish resulting from his operations.

DATA ON STRUCTURAL STEEL *

Estimating the Cost of Structural Steel for Buildings

Structural steel for buildings is commonly made up of I beams, chann angles, Z bars and plates, which may be used as single beams or braces, or bu into riveted girders, columns, or trusses. The Z bars are now seldom used columns or other structural work in buildings. The cost of the completed ste work is made up of the following items:

(1) Cost of the plain steel at the mill, plus freight and dealers' profits.

(2) Extras for cutting, punching, fitting and assembling into girders, colum or trusses.

(3) Cost of the fittings, such as connection-angles, gusset-plates, etc.

(4) Shop-painting.

(5) Cost of erection at the building.

(6) Painting after erection.

Base-Price of Steel. For orders of any considerable size, the cost of pl steel is based on the price at the mills plus the freight to the point of deliver

The BASE-PRICE, free on board cars at Pittsburgh, Pa. (1920), is ab $2.45 per 100 lb for I beams and channels 15 in and less, and for angles zees from 3 to 6 in.

I beams over 15 in, cost 10 cts per 100 lb extra, and tees over 3 in, 5 cts ex

For angles, channels and zees under 3 in, the base is $2.45 at Pittsburgh.

For angles, over 6 in, $2.45 + $0.10.†

For H beams, $2.45 + $0.10.

For deck beams and bulb angles, $2.45 + $0.30. ‡

For corrugated and checkered plates, $2.65 + $1.75.§

For plates, structural, the base is $2.65.

* Valuable data was contributed for this section by Associate Editor, Robins Flem

† $2.45 + $0.10 means a base-price of $2.45 and an extra $0.10.

‡ $2.45 + $0.30 means a base-price of $2.45 and an extra of $0.30.

§ 2.65 + $1.75 means a base-price of $2.65 and an extra of $1.75; the same with $ + $0.15, etc. Corrugated steel, painted, is usually quoted at a base-price plus an e for painting. At present (1920) it is $4.25 + $0.25.

ates, flange, the base is $2.65 + $0.15.*
orrugated steel, painted, No. 22, $4.25 + $0.25.*
orrugated steel, galvanized, No. 22, $5.30.
eel sheets, black, Nos. 10 and 11, $4.00.
eel sheets, galvanized, Nos. 10 and 11, $4.70.
eel sheets, black, No. 22, $4.20.
eel sheets, galvanized, No. 22, $5.25.
r-iron, the base is $4.50.
vets, $4.50.
eel bars, $2.35.

t-Rates (March, 1920) in car-load lots are:

gh to Albany, N. Y.	27.0 cts
to Baltimore	23.0 cts
to Boston	29.5 cts
to Buffalo, N. Y.	21.0 cts
to Chicago	27.0 cts
to Cincinnati	23.5 cts
to Cleveland	17.0 cts
to Columbus, O.	20.0 cts
to Denver	99.0 cts
h to Louisville	26.5 cts
to New York	27.0 cts
to Norfolk, Va	31.5 cts
to Philadelphia	25.0 cts
to Richmond, Va.	30.0 cts
to Rochester, N. Y.	21.0 cts
to St. Louis	34.0 cts
to Washington, D. C.	24.0 cts

unt of the expense of carrying beams in stock, local dealers usually n ½ to 1½ ct a pound, extra, on orders supplied from stock.†

rd Classification of Extras. These lists are for STEEL BARS AND APES, and the extras are added ,to the BASE-PRICES for each 100 This standard classification was adopted June 15, 1919, by the teel Company.

Specification and Inspection

ial, subject to United States Navy Department specifications edium or soft steel	$0.10
e hull-steel (except rivet-rods) subject to United States Navy tment specifications	1.00

for other than mill-inspection, such as Lloyd's or American Bureau , for buyer's account.

Quantity-Differentials

fications for less than 2 000 lb of a size will be subject to the fol- is, the total weight of a size ordered to determine the extra, regard- h and regardless of exact quantity actually shipped:

$1.75 means a base-price of $2.65 and an extra of $1.75; the same with , etc. Corrugated steel, painted, is usually quoted at a base-price plus painting. At present (1920) it is $4.25 + $0.25.
t (1920) a war tax of 3% is to be added to the rates given.

Quantities less than 2 000 lb, but not less than 1 000 lb..................
Quantities less than 1 000 lb..

Straightening

Machine-straightening...

Machine-Cutting to Specified Lengths, Rounds and Squares, 1½ Inches ... Larger

Machine-cutting to lengths over 48 in.......................................
Machine-cutting to lengths over 24 in to 48 in, inclusive..............
Machine-cutting to lengths over 12 in to 24 in, inclusive..............
Machine-cutting to lengths of 12 in and less, extra will be furnished on application, but will not be less than...........................

The above extras apply only to .50 per cent carbon and under. Extras ... machine-cutting over .50 per cent carbon will be furnished on application.

Extras for machine-cutting Rounds and Squares under 1½ in, Flats, ... will be furnished on application.

Cutting to Specified Lengths, Other than Machine-Cutting

Cutting to lengths of 60 in and over...............................No ...
Cutting to lengths over 48 in to 59 in, inclusive.......................
Cutting to lengths over 24 in to 48 in, inclusive.......................
Cutting to lengths over 12 in to 24 in, inclusive.......................
Cutting to lengths of 12 in and less, extra will be furnished on application, but will not be less than......................................

Cost of Erecting. For erecting ordinary beams and columns in build... having masonry walls the cost of erection should not exceed $20 per ton ... there are bolted connections, and it will sometimes be as low as $13 per ton. ... erecting the steelwork of skeleton buildings having riveted connections ... customary to allow $18 per ton.

Cost of Painting. The usual charge for shop-painting is about $3 ... ton, but if done in accordance with the specification on page 1203 it would ... this amount. For painting one additional coat after erection, allow about ... per ton.

Roof-Trusses. In lots of at least six, the shop-cost of ordinary roof-tr... in which the ends of the members are cut off at right-angles is about as follo... Trusses weighing 1 000 lb each, from $2.00 to $3.50 per 100 lb; trusses weigh... 1 500 lb each, from $2.00 to $2.50 per 100 lb; trusses weighing 2 500 lb ... from $1.50 to $2.50 per 100 lb; and trusses weighing from 3 500 to 7 50... from $1.25 to $2.00 per 100 lb. Pin-connected trusses cost from 10 to 30 c... 100 lb more than riveted trusses.*

Steel Mill-Buildings. The average shop-cost for the frames of steel ... buildings, including draughting, is about $40 per ton, and the cost of erec... from $20 to $35 per ton.*

Cost of Drafting. Details for church and court-house roofs having ... and valleys cost from $10 to $20 per ton; details for ordinary mill-building... from $6 to $12 per ton. The cost of making shop-drawings varies greatly ... the character of the construction of the buildings, and with the accuracy o...

* If there is little duplication or parts of if manual labor enters into the fabrica... any great extent the costs given will be increased.

's drawings. The average costs per ton of steel, for making shop- are about as follows:

tire skeleton construction, in which the loads are all carried to the undations by the steel columns, $4.00.
e interior parts which are supported on steel columns, when the outside lls carry the floor-loads and their own weight, $3.50.
e interior parts which are supported on cast-iron columns, when the tside walls carry the floor-loads and their own weight, $2.50.
istruction without columns, and in which the floor-beams rest on ma- ry walls, $2.50.
ldings in which roof-trusses supported by columns comprise the greater t of the construction, $7.00.
ldings in which roof-trusses on masonry walls comprise the greater t of the construction, $4.00.
l-buildings, average, $9.00.
iufacturing or shop-buildings, with flat roofs, and one story in height, o.
rations, additions, remodeling, which require measurements before ils and shop-drawings can be made, $12.00.*

imate Estimates of the Weight of Steel in Buildings. Accord- G. Tyrell,† the weight of steel in any proposed new building may be imated from the following data, which is a fair average for buildings leven stories high, designed according to the Building Laws of the ston:

	Per sq ft of floor
nent-houses and hotels, with outside frame	14 lb
nent-houses, without outside frame	9 lb
uildings, with outside frame	23 lb
uildings, without outside frame	15 lb
uses, with outside frame	28 lb
uses, without outside frame	18 lb

ings higher than eleven stories, the weight of floors will increase in rtion to the number of stories, while the weight of columns will e rapidly.
pproximate weight of roof-trusses, see Chapter XXVII, pages 1050

Weights of Steel in Buildings ‡

ffecting the Weights of Steel Structures are many and varied. er square foot of area or per cubic foot of volume of a structure should not be assumed as the weight of a proposed structure conditions which govern the one are found in the other. Munici- codes specify floor-loads and these vary greatly. The prescribed , working stresses and column-loads, affect the weight. The features to be followed also play an important part. In mill- weight is affected by the kind of roofing and siding used, capacity

f $12.00 includes the cost of taking measurements. This generally has to contractor.
Structural Steel, in Architects & Builders' Magazine, Jan., 1903.
by Robins Fleming.

of cranes, spacing of trusses and columns, shafting, special loadings and allowable minimum thickness of metal.

Weights of Steel in a Number of Structures are given in the follow table and notes. The caution regarding such weights being taken as pre dents should be emphasized. The office-building heading the list is Equitable building, the largest office-building in the world.

	Average dimensions in feet			Tiers of beams	Weight in pounds per square foot of framed area	Weight pounds cubic f of volum
	Width	Length	Height			
Office-buildings	159	308	542	41	37.00	2.55
	43	79	217	17	26.10	2.2[illegible]
	90	90	258	22	28.92	2.4[illegible]
	81	139	225	19	21.90	1.8[illegible]
	43	104	149	13	33.40	2.9[illegible]
	48	111	115	9	17.34	1.51
Hotels	97	119	244	20	26.02	1.92
	84	143	270	24	26.95	2.39
	96	101	232	18	25.40	2.0[illegible]
	108	120	115	9	14.00	1.0[illegible]
Department-stores	133	219	150	11	23.87	1.7[illegible]
	62	211	130	8	29.44	1.8[illegible]
	103	132	89	7	18.30	1.4[illegible]
Warehouses	100	105	131	10	22.85	1.7[illegible]
	88	121	121	9	20.60	1.8[illegible]
	145	357	102	7	30.80	2.1[illegible]
	58	72	52	3	31.35	1.8[illegible]

Among prominent New York buildings the 55-story Woolworth Bui with a ground-area of 31 000 sq ft weighs 3.0 lb per cu ft.; the 39-story Ban Trust Building with an area of 9 000 sq ft, 3.1 lb.; the 25-story Muni Building with an area of 42 700 sq ft, 3.6 lb; the 25-story Hotel McA with an area of 31 000 sq ft, 2.0 lb. The 10-story Curtis Building of Ph delphia with an area of 94 000 sq ft weighs 3.0 lb. The structural ste four buildings of Pittsburgh, the Arrott, the Farmers' Bank, the Empire the Oliver, is quoted as weighing respectively 2.8, 2.3, 2.1 and 1.8 lb per c For buildings of from 8 to 12 stories in which the exterior walls are carrie steel framing the weight per cubic foot of volume may be assumed at 1 for office-buildings and 1.5 for hotels.

Armories. The three-hinged arches with roof-framing of an armor Brooklyn, 191 by 300 ft in area, weighs 15.5 lb per sq ft of ground area. armory in Buffalo, 233 by 335 ft, weighs 18.3 lb. The steelwork of the K bridge Armory, New York City, 289 by 590 ft, said to cover the largest hall in the world, weighs about 90 lb per sq ft, of which one half is roof one half floor and miscellaneous framing.

Boiler-Shops. Sizes and weights per square foot of a few boiler-shop as follows: 167 by 336 ft, three aisles, floor in center and cranes in outer concrete roof and sides, steel purlins and girts, 23.9 lb; 124 by 300 ft,

h 15, 25 and 50-ton cranes respectively, steel purlins and brick walls :olumns, 36 lb; 74 by 160 ft, 10-ton crane in center aisle, single beams aisles to carry roof, galvanized corrugated-steel covering and siding, 85 by 140 ft, two aisles, one with crane, 20.8 lb; 94 by 97 ft, two aisles, :rane, 26.3 lb.

ırns. The steel roof-trusses and bracing of a car-barn 100 by 154 ft, ins, brick walls, weighs 6.2 lb per sq ft. Another car-barn, 44 by 270 ft. l-steel roof, and sides on steel purlins and girts, 9.15 lb. Another, 4 ft, four aisles, concrete roof on steel purlins, 11.8 lb.

t-Plants. Four cement-plants with ground-areas of 58 000, 73 000, l 128 000 sq ft respectively, weigh respectively, 23.6, 22.0, 23.5, and These weights are the averages of the buildings that usually form a nt. The individual buildings vary from 10 lb for an engine-room for a clinker-grinding room.

ınkers. The weights of six coal-bunkers of the suspended type and ities of from 350 to 1 000 tons, range from 128 to 234 lb per ton of he average being 204 lb. A system of rectangular pockets to store (10 ft 6 in from ground to valves) weighs 158.3 lb per ton of capacity. the weights of supports but not of roofs are included. A 35 by 70-ft pported on plate girders with a capacity of 1 000 tons weighs 240 lb capacity, including the roof-trusses that carried the conveyor.

ıops. The steel framing for the roof of a forge-shop 83 by 126 ft, ımns and no cranes, covered with corrugated steel on steel purlins, lb per sq ft of ground-area. A forge-shop 220 by 240 ft, four aisles, rane-runways, composition roofing, concrete sides, steel purlins and 24.6 lb. A forge-shop 110 by 425 ft for heavy work, 47 ft 6 in to rd, two aisles each with a 50-ton crane, tile roof, glass and brick 40 lb.

s. A pipe-foundry, 50 by 150 ft, slate covering, wooden purlins, 15-ton crane, weighs 11.35 lb per sq ft. A similar one for the ny, 45 by 82 ft, with a 30-ton crane, weighs 17.23 lb. A foundry, , one center aisle, with light crane, lean-to each side, corrugated-d sides, weighs 14.8 lb. A foundry, 150 by 290 ft, for a pump-ur aisles with 20-ton crane in one aisle, wooden purlins, two charging-floors of concrete on steel beams, weighs 13.9 lb. A by 252 ft, equipped for heavy work, 60-ft center aisle, two side charging-floor, storage-platform, weighs 38.9 lb.

Shops. A machine-shop, 90 by 328 ft, for heavy work, one o ft wide with 25-ton crane, each side aisle 25 ft wide with gallery-on crane underneath, tile roof on steel purlins, brick and glass 43 lb per sq ft of ground-area. A two-story machine-shop, three aisles, light cranes in lower story, composition roof, steel ete sides, weighs 35.15 lb. A one-story building, 75 by 300 ft, om chord, shafting, corrugated-steel roofing and siding, weighs ther one-story building, 70 by 100 ft, 18 ft to bottom chord, rete roof on trusses 10 ft apart, no purlins, weighs 13.88 lb. In steel framing for the Hy-rib sides of this building weighs 3.44 lb ertical surface. A machine-shop, 116 by 252 ft, 60 ft center aisle, o-ton-crane runway and lower 25-ton-crane runway, two side le with traveling jib-cranes, weighs 33 lb.

lls. A rolling-mill, 93 by 186 ft, corrugated-steel roof and 7.6 lb per sq ft. Another, 170 by 384 ft, two aisles each with

5-ton cranes, saw-tooth roof-trusses on longitudinal girders, concrete slab steel purlins, brick walls between columns, weighs 17.5 lb. A similar buil for shop-purposes weighs 18.62 lb.

Paper-Mills. The entire structural steel for three paper-mills we respectively 18.4, 20.6 and 21.4 lb per sq ft of area. All roof-trusses ar the flat type, spaced 8 ft apart in the first and third, and 16 ft in the second

Power-Houses. A power-house, 44 by 186 ft, 49 ft to bottom chord, ton crane, tile roof on steel purlins, brick walls between columns, weighs per sq ft. Another, 53 by 270 ft, 33 ft to bottom chord, 20-ton crane, tile on steel purlins, brick walls and sash between columns, weighs 39.6 lb. Anot 120 by 96 ft, one aisle for boiler-room and one with 10-ton crane for eng room, steel purlins for concrete roof-covering, brick walls between colu weighs 17.8 lb.

Train-Sheds. The train-shed of the Pennsylvania Railroad in P delphia, 598 ft long and with arches 300 ft 8 in from center to center of weighs 39.1 lb per sq ft of ground-area; the train-shed of the same railroa Jersey City, 777 ft long and with arches 252 ft 8 in, weighs 27.9 lb; and th the Philadelphia & Reading Railroad in Philadelphia, 506 ft 8 in long with arches 259 ft 8 in, weighs 31.5 lb. The train-shed, 390 by 815 ft, of Central Railroad of New Jersey in Jersey City, is a series of concrete and umbrellas, of the Bush-type. The structural steel weighs 17 lb per sq ft of a

Three Industrial Plants. In one of the plants of a great industrial poration a two-story shop, 51 by 380 ft, weighs 28 lb per sq ft of ground- a three-story shop, 80 by 420 ft, 37.9 lb; a three-story shop, 80 by 300 ft, 46. a three-story shop, 80 by 630 ft, 67.5 lb; a four-story shop, 77 by 14 66.6 lb; a foundry, 121 by 150 ft, 40.5 lb. In another plant of the same poration, a three-story machine-shop, 80 by 510 ft, weighs 84.3 lb; a five- office-building, 49 by 243 ft, 70.3 lb; a power-house, 55 by 120 ft, 37. a blacksmith-shop, 81 by 200 ft, 15.6 lb. In a plant of another corpora a boiler-house, 50 by 94 ft, weighs 23.3 lb; a furnace-building, 60 by 16 25.1 lb; a rolling-mill, 80 by 80 ft, 24.4 ft; a rod-mill, 243 by 220 ft, 28.1

Cost of Merchant Steel. The cost of merchant iron and steel of all k is based on a certain size of each particular shape, which is taken as the and the price of all other sizes is figured at a certain extra rate above the according to a standard CARD OF MILL-EXTRAS. The BASE-PRICE may tuate and be changed without notice, but the extras remain constant, and the same in all localities. The following tables include the standard classific of extras on iron and steel bars.

Standard Classification* of Extras on Iron and Steel Bars
Adopted July 15, 1919.

Rounds and squares

Sizes	Extra per 100 lb	Sizes	Extra per 100 lb
6 in	Base	7/32 in	$1.00
6 in	$0.05	3/16 in	1.25
in	0.10	3 1/8 to 3 9/16 in	0.075
in	0.20	3 5/8 to 4 1/16 in	0.125
in	0.25	4 1/8 to 4 9/16 in	0.15
in	0.30	4 5/8 to 5 1/16 in	0.20
in	0.35	5 1/8 to 5 9/16 in	0.25
in	0.40	5 5/8 to 6 1/16 in	0.375
in	0.50	6 1/8 to 6 1/16 in	0.50
in	0.75	6 5/8 to 7 1/4 in	0.625

Flats

Sizes	Extra per 100 lb
in × 1 3/8 to 1 in	Base
in × 1/4 to 5/16 in	$0.10
5/16 in × 3/8 to 3/4 in	0.20
/16 in × 1/4 to 5/16 in	0.25
in × 3/8 to 1/2 in	0.25
in × 1/4 to 5/16 in	0.35
in × 3/8 to 7/16 in	0.50
in × 1/4 to 5/16 in	0.60
in × 3/8 in	0.70
in × 1/4 to 5/16 in	0.80
in × 1/4 to 5/16 in	1.00
in × 1 1/16 to 1 3/16 in	0.05
in × 1 1/4 to 1 1/2 in	0.10
in × 1 5/8 to 2 3/4 in	0.15
in × 3 to 4 in	0.20

Standard Classification † of Angles, Channels and Tees

Angles

Sizes	Extra per 100 lb
and wider, but under 3 in × 3/16 in and over	$0.10
and wider, but under 3 in × 1/8 in	0.15
1/4 × 1 1/4 in × 3/16 in and over	0.15
1/4 × 1 1/4 in × 1/8 in	0.20
in × 3/16 in	0.20
in × 1/8 in	0.25
in × 3/16 in	0.25
in × 1/8 in	0.30
in × 1/8 in	1.10
in × 3/32 in	1.30
in × 1/8 in	1.60
in × less than 1/8 in	1.80
both legs × less than 1/4 in	0.35

ngles are subject to special prices, which will be furnished on application

e sizes take the next higher extra. It is not customary to enforce mor
e "standard-card extras" for round and square bars.
e sizes take the next higher extra.

Standard Classification * of Angles, Channels and Tees (Concluded)

Channels

Sizes	Extra pe 100 lb
1½ in and wider, but under 3 in × 3/16 in and over	$0.15
1½ in and wider, but under 3 in × 1/8 in	0.25
1 to 1¼ in × 3/16 in and over	0.25
1 to 1¼ in × 1/8 in	0.35
1 to 1¼ in × 7/64 in	0.50
¾ and ⅞ in × 3/16 in and over	0.30
¾ and ⅞ in × 1/8 in	0.40
¾ and ⅞ in × 7/64 in	0.55
⅝ in × ⅛ in and over	1.20
⅝ in × 3/32 in	1.40
½ in × 7/64 in and over	1.80
½ in × 5/64 in	2.00

Tees

Sizes	Extra pe 100 lb
1½ × 1½ in and wider, but under 3 in × 3/16 in and over	$0.20
1 × 1 to 1¼ × 1¼ in × 3/16 in and over	0.40
1 × 1 to 1¼ × 1¼ in × ⅛ in	0.50
⅞ × ⅞ in × 3/16 in	0.50
⅞ × ⅞ in × ⅛ in	0.60
¾ × ¾ in × 3/16 in	0.60
¾ × ¾ in × ⅛ in	0.70
⅝ × ⅝ in × ⅛ in	1.30
½ × ½ in × ⅛ in	1.80
Unequal-leg tees are subject to special prices, which will be furnished on application.	

* Intermediate sizes take the next higher extra.

The base for car-load lots for any city may be obtained by adding the frei rates given on page 1524 to the base prevailing at the mills.

CHAPTER XXXI

)MICAL AND VAULTED STRUCTURES *

By

EDWARD F. RIES

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1. Domes *

cation: Domical structures may be considered under two main (1) Smooth-shell domes, and (2) ribbed domes. The first division be divided into (a) domes with shells of uniform thickness, and (b) h shells of uniformly varying thickness. The materials of con- f division (1) are brick, stone, concrete, and tile; and of division (2), rete, and wood. A dome may be constructed with or without a ›r with or without an OCCULUS or EYE; and in the case of ribbed r may have either circular or polygonal bases.

(1) Smooth-Shell Domes

ical Principles. Under this heading are considered both (a) domes of uniform thickness, and (b) domes with shells of uniformly vary- ss, and also domes with or without lanterns and eyes. A dome tapers toward the top is the more stable dome. It is evident that urt, or crown, tends to fall in and thereby push out the lower por- the lighter the upper part is in relation to the lower part, the the dome. The exact actions of the INTERNAL STRESSES in a dome o determine, but a very practical solution can, however, be developed ng that the stresses are parallel to a surface midway between the ner surfaces of the dome.

Analysis. A dome may be imagined to consist of a number of ngs of decreasing diameter, each one laid on top of another. er part tends to fall in and push out the lower part, there must be a contract each ring in the upper part and to expand each ring in the That is, there must be END-COMPRESSION on all stones (imaginary oncrete) of the upper part, and END-TENSION on all stones of the The dividing line or horizontal joint between these upper and lower dome is called the JOINT OF RUPTURE. The angle made by the ıre with the vertical (center of dome as apex of angle) is known CAL ANGLE. It is evident, then, that the determination of the TURE and the CRITICAL ANGLE determines also the points below s tension in the rings. By reinforcing the lower part with steel to resist this tension, the dome can be made secure. If the dome E, that is, one in which the angle the base makes with the vertical he CRITICAL ANGLE, the tension-steel must be placed at the base › resist the outward push or thrust,

* See, also, Chapters VII and VIII.

Notation and Theory (See Fig. 1):

r = mean radius of dome;
a = thickness of shell at crown;
t = thickness of shell or of ring at any point;
α = angle made with the vertical by radius to lantern-ring. Center of dome is the apex of angle. In a dome without lantern, $\alpha = 0$;
θ = angle made with the vertical by radius passing through any point in shell (in equations angles are in radians);
c = constant of variation of t with respect to arc θ;
$\frac{cr}{a}$ = a constant (based on above notation) for any dome;
ϕ = critical angle, that is, the angle made with the vertical by the joint of rupture;
w = weight of cubic unit of masonry;
V = volume of shell of complete dome above any ring;

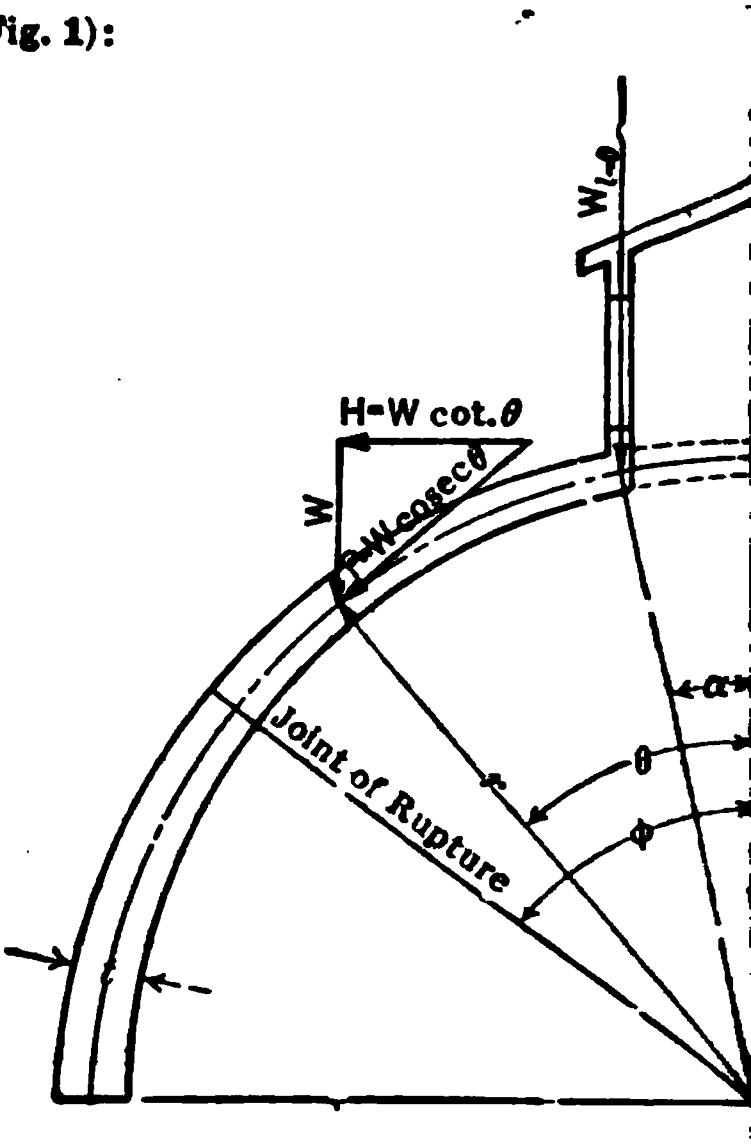


Fig. 1. Smooth-shell Concrete Dome. Analysi

$W_d = wV$ = total weight of complete shell above any ring (including the removed for eye);
W_{l-o} = weight of lantern minus weight of shell removed for occulus or (${W_{l-o}}$ may be either positive or negative);
$W = W_d + W_{l-o}$;
$n = \frac{W_{l-o}}{2\pi w a r^2}$, a constant for any dome;
P = total tangential pressure for any ring, due to lantern and shell a that ring;
U = tangential pressure per unit-length of ring;
H = total radial horizontal pressure on any ring, due to outward pus thrust of shell above that ring;
T = hoop-tension or hoop-compression in ring, due to H;

Using this analysis and notation, the following equations are developed:

Equation (1) $t = a + cr\theta$

Equation (2) $V = 2\pi r^2[a(1 - \cos\theta) + cr(\sin\theta - \theta\cos\theta)]$

Equation (3) $W_d = wV = 2\pi w a r^2\left[(1 - \cos\theta) + \frac{cr}{a}(\sin\theta - \theta\cos\theta)\right] =$

$$s = 2\pi\left[(1-\cos\theta)+\frac{cr}{a}(\sin\theta-\theta\cos\theta)\right]$$

n (4) $$P = 2\pi w a r^2\left[(n \operatorname{cosec}\theta + \operatorname{cosec}\theta - \cot\theta)+\frac{cr}{a}(1-\theta\cot\theta)\right]$$

n (5) $$U = w a r\left[\frac{n}{\sin^2\theta}+\frac{\operatorname{cosec}\theta-\cot\theta+\frac{cr}{a}(1-\theta\cot\theta)}{\sin\theta}\right]$$

$$= w a r\,(S_1+S),\quad \text{in which}$$

$$S_1 = \frac{n}{\sin^2\theta}\quad\text{and}$$

$$S = \frac{\operatorname{cosec}\theta-\cot\theta+\frac{cr}{a}(1-\theta\cot\theta)}{\sin\theta}$$

(6) $$T = \frac{H}{2\pi} = w a r^2\left[n\cot\theta+(1-\cos\theta)\cot\theta + \frac{cr}{a}(\sin\theta-\theta\cos\theta)\cot\theta\right]$$

$$= w a r^2\,(Y_1+Y),\ \text{in which}$$

$$Y_1 = n\cot\theta\quad\text{and}$$

$$Y = \left[(1-\cos\theta)\cot\theta+\frac{cr}{a}(\sin\theta-\theta\cos\theta)\cot\theta\right]$$

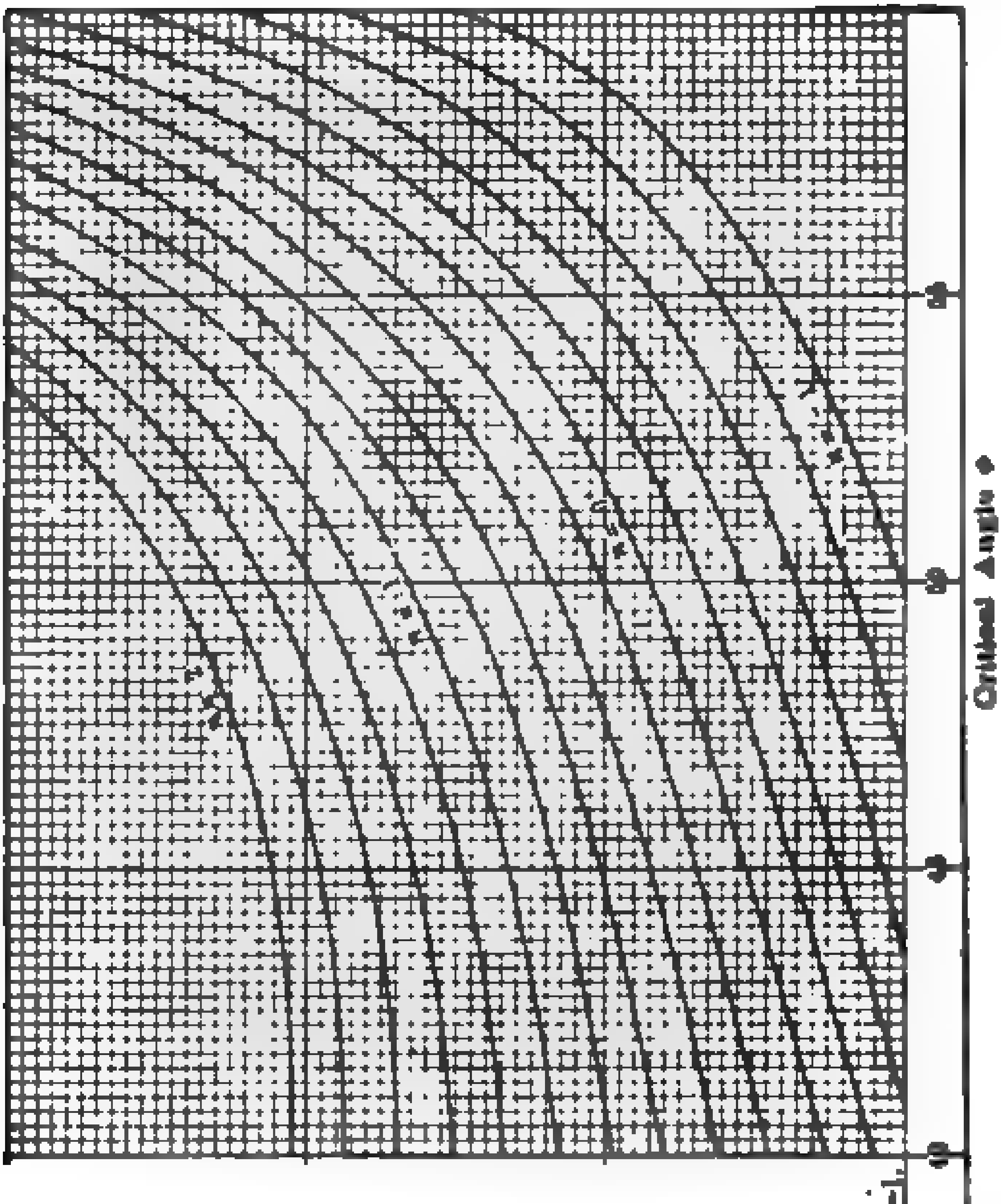
7) $$\frac{cr}{a} = \left[n\operatorname{cosec}^2\theta-\frac{\cos\theta-\sin^2\theta}{1+\cos\theta}\right]+\left[\theta\cos\theta-\frac{1-\theta\cot\theta}{\sin\theta}\right]$$

and Investigation of Smooth-Shell Circular Domes. By the use
going equations any CIRCULAR DOME can be designed or investigated.
tations, however, connected with some of these equations are long
, and are simplified by using curves plotted from the solutions
giving different values to some of their elements or factors. (See
III, and IV.)

n (7) is represented by the curves in Plate I. By the use of these
osition of the JOINT OF RUPTURE for any dome is found by inspection
ues of $\frac{cr}{a}$ and n are known. The value of $\frac{cr}{a}$ is easily determined, as c
by using Equation (1) after determining or assuming a, the thick-
rown, and t, the thickness at the base; and the value of n is found
$n = \frac{W_{l-o}}{2\pi w a r^2}$.

3) is represented by the curves in Plate II. From these curves the
shell is determined.

5) is represented by the curves in Plate III. Knowing the values
e values of S_1 and S are found by inspection, and hence U is easily



Values of $\frac{\sigma}{s}$

Plate I. Curves for Determination of Joint of Rupture of Domes. Based on Equation (

Equation (6) is represented by the curves in Plate IV. Knowing the values
n and $\frac{\sigma}{s}$, the values of Y_1 and Y are found, and T computed for any ring. Wh
n equals zero, Y_1 equals zero, and the value of T depends upon Y as given in t
lower curves. It will be noticed that Y increases as θ increases until the cri
ical angle for a dome without lantern, or eye ($n = 0$), is reached; that is, ea
successive ring increases the outward thrust, and at the CRITICAL ANGLE there
a maximum value of Y, and hence a MAXIMUM HOOP-TENSION T. After t
CRITICAL ANGLE is passed the rings are in tension, and therefore T and Y a
reduced by the tension required of the ring or masonry.

The curves also indicate that the stability of a dome with a shell of unifo

Angle θ

Values of z

· **Determination** of Weight of Shell of Domes. Based on Equation (3)

no lantern is not affected by the thickness of the shell,

refore $\frac{cr}{a} = 0$, regardless of the value of a.

equired to design a SMOOTH SHELL REINFORCED CONCRETE and with a lantern of 10-ft radius, weighing 50 000 lb. The ern is to be removed, forming an eye. (Fig. 2.)

a crown-thickness, a, of 5 in, and a thickness, t, at the base

$$t = a + cr\theta, \text{ or } \frac{cr}{a} = \frac{t - a}{a\theta}$$

Angle θ

Plate III. Curves for Determination of Tangential Stress per Unit-length of De ring. Based on Equation (5)

For the dome without wind-loads or snow-loads

$$\frac{\sigma}{a} = \frac{\frac{8}{12} - \frac{5}{12}}{\left(\frac{5}{12}\right) 1.396} = 0.43$$

The angle $\alpha = \sin^{-1} \frac{10}{45} = 12° \ 50'$.

From Plate II the weight of the shell removed for the eye is

$$150 \left(\frac{5}{12}\right) (45)^2 (0.165) = 20\ 883 \text{ lb}$$

Angle θ

'urves for Determination of Hoop-tension or Hoop-compression in Dome-ring. Based on Equation (6)

$$W_{t-\theta} = 50\,000 - 20\,883 = 29\,117 \text{ lb}$$

loads and snow-loads a simple and safe method of procedure is to ›rm load over the surface of the dome, since this load can be trans- equivalent in inches of masonry and hence the same equations and A wind-load, for example, of 25 lb per sq ft, is equivalent to 2 in veighing 150 lb per cu ft. Hence the new a and t equal 7 and 10 in, Hence from Equation (1)

$$\frac{\sigma}{a} = \frac{\frac{10}{12} - \frac{7}{12}}{\left(\frac{7}{12}\right) 0.1396} = 0.307$$

and

$$n = \frac{W_{l-o}}{2\pi w a r^2} = \frac{29\,117}{2\pi(150)\left(\frac{7}{12}\right)(45)^2} = 0.026 \text{ *}$$

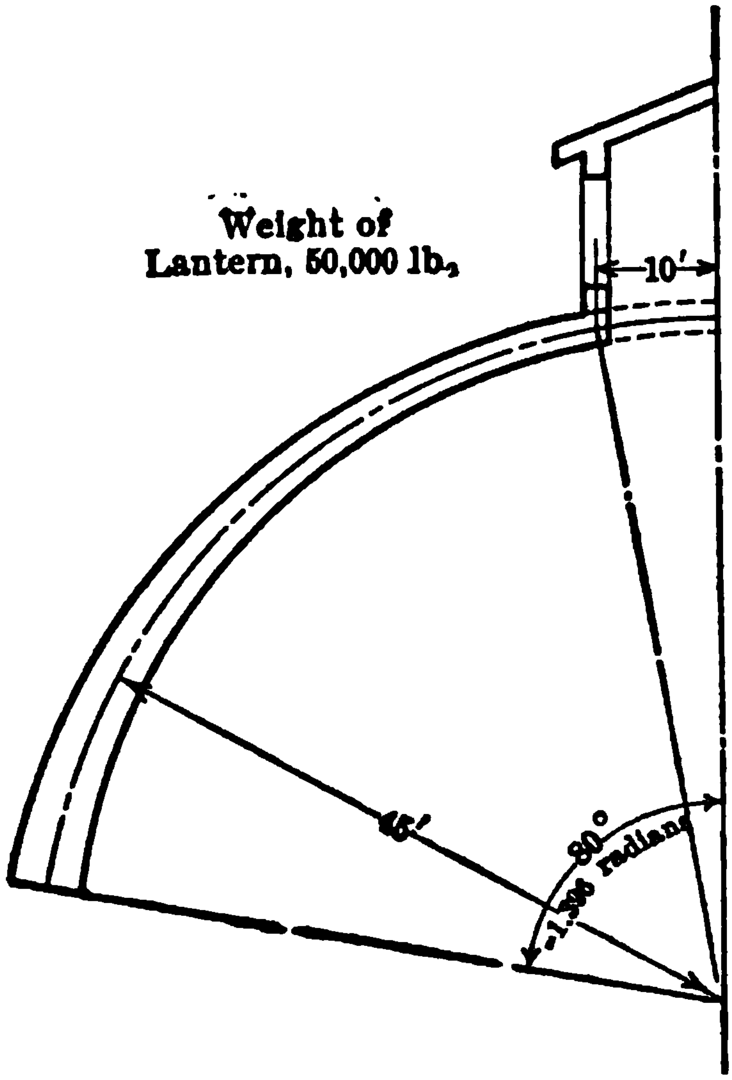


Fig. 2. Smooth-shell Concrete Dome with Lantern and Eye. See Example

From Plate I, with $\frac{cr}{a} = 0.$
and $n = 0.026$, the CRITICAL AN
is found to be 52° 35'.

From Plate IV, at the CRIT
ANGLE for the dome with sn
load and wind-load

$$T = 150\left(\frac{7}{12}\right)(45)^2(0.020 + 0.$$
$$= 65\,914 \text{ lb tension}$$

This must be resisted by
reinforcing rods. Allowing a
tensional stress of 16 000 lb
sq in in the steel, a total
$\frac{65\,914}{16\,000} = 4.12$ sq in sectional
of steel is required. At the b
($\theta = 80°$)

$$T = 150\left(\frac{7}{12}\right)(45)^2(0.005 + 0.$$
$$= 33\,843 \text{ lb}$$

The total cross-sectional are
steel in tension at the bas
$\frac{33\,843}{16\,000} = 2.12$ sq in, given by
round rods, each ¾ in in diam

The remaining required sectional area of steel, $4.12 - 2.12 = 2$ sq in,
be spaced in the lower part of the dome over an angular distance of (80° — 52°
= 27° 25' = 0.4785 radian, or $0.4785 \times 45 = 21.53$ ft up the surface of
dome.

The assumed thickness of shell at the base was 8 in, and the thickness at
lantern-ring will be, from Equation (1),

$$t = a + cr\theta = a + a\frac{cr}{a}\theta$$

or

$$5 + 5\,(0.43)(0.2239) = 5.48 \text{ in}$$

Allowing 0.2 per cent of steel cross-section, horizontally and meridionally,
SECONDARY STRESSES caused by temperature-changes and possible une
snow-loads and wind-loads, there should be $8 \times 12 \times 0.002 = 0.19$ sq in of
cross-section per running foot at the base, and $5.48 \times 12 \times 0.002 = 0.13$ s
per running foot at the lantern-ring. The spacing of the horizontal reinfo

* The snow-load on the top of the lantern is taken care of because snow-loads and w
loads over the entire dome were included, and only the actual masonry of the eye
subtracted.

ound as indicated in Fig. 3. Curve *A* gives the total amount of steel *r* for SECONDARY STRESSES above any point in the cross-section of the urve *B* gives the necessary tensional resistance, using Curve *A* as a

ram for Determination of Amount of Horizontal Steel Reinforcing in Concrete Domes

for the ordinates. Various points on the curve are easily deter- ample, for $\theta = 70°$

$$t = 5 + 5(0.43)(1.2217) = 7.63 \text{ in}$$

n of temperature-steel per foot at 70° is

$$7.63 \times 12 \times 0.002 = 0.18 \text{ sq in}$$

ection of temperature-steel above 70° is then

$$\frac{0.18 + 0.13}{2} \times (1.2217 - 0.2239)45 = 6.96 \text{ sq in}$$

cross-section (Plate IV) above 70° is

$$\frac{(150)\ (7/12)\ (45)^2(0.298 + 0.009)}{16\,000} = 0.72 \text{ sq in}$$

ermined in this manner the curves are developed. section of horizontal steel in the entire cross-section of the

$$8.45 + 2.00 = 10.45 \text{ sq in}$$

2.12 sq in of tension-steel at the base. If ½-in round rods be $\frac{10.45}{0.1963} = 54$, required in the shell. By dividing the area

below the curve (Fig. 3) into 54 parts, the distance up from the base, where eac rod should be placed, is determined. The meridional steel should be such th there will be 0.19 sq in of cross-section per foot of circumference at the bas and 0.13 sq in per foot of circumference at the lantern-ring; that is, if ½-in roun rods are used, they should be spaced $\frac{0.1963}{0.19} = 1.03$ ft at the base, and $\frac{0.1963}{0.13}$ 1.51 ft at the lantern-ring. The punching-shear at the lantern-ring is equal t

$$\frac{50\,000}{(5.48)\,(2\pi 10 \times 12)} = 12.1 \text{ lb per sq in}$$

This is well within the limit of 40 lb per sq in.

(2) Ribbed Domes

General Principles. The following discussion applies to domes of eith circular or polygonal horizontal cross-sections. All steel domes are ribb domes, and usually have from six to twenty-four ribs resting against a LANTER RING OR SPIDER at the top. The ribs may have solid webs, perforated webs, latticed webs, with angle or channel-flanges. The latticed angle-ribs are prefe able because of their lightness. The tension-rings and compression-rings m be built similar to the main ribs, and should brace the latter through rig gusset-connections. The diagonals are usually rods with turnbuckles for adjus ment. CONCRETE-RIBBED DOMES or WOODEN-RIBBED DOMES may be design according to the same general principles followed for steel domes, but the dia onals are omitted and dependence for rigidity is placed on the slab-fillin between the ribs.

The Schwedler Method for the Design of Steel Domes. W. Schwed has by simple resolution of the forces derived equations for domes, based on forms of SURFACES OF REVOLUTION. These equations are easily checked wh the forces acting through a RIB (the rib acting as a strut between the joint and through a RING at a joint are considered. The following laws may stated:

(1) The ribs are in maximum stress when the whole dome is loaded;

(2) A ring is in maximum tension when all of the dome above the ring is fu loaded, and in maximum compression when all of the dome below the ring an the ring itself is fully loaded;

(3) The DIAGONALS are not stressed when the dome is symmetrically load The diagonals in a panel are in maximum stress when the dome on one side of meridional plane passed through the center of that panel is fully loaded an the other side unloaded.

In Fig. 4 let

α_1, α_2, α_3, etc. = angles made by rib-sections with the horizontal;

β_1, β_2, β_3, etc. = angles made by diagonals with the ribs;

P_1, P_2, P_3, etc. = dead loads at ends of rib-sections;

L_1, L_2, L_3, etc. = live loads at ends of rib-sections;

D_1, D_2, D_3 etc. = stresses in rib-sections;

T_1, T_2, T_3, etc. = stresses in rings;

N_1, N_2, N_3, etc. = stresses in diagonals;

n = number of ribs.

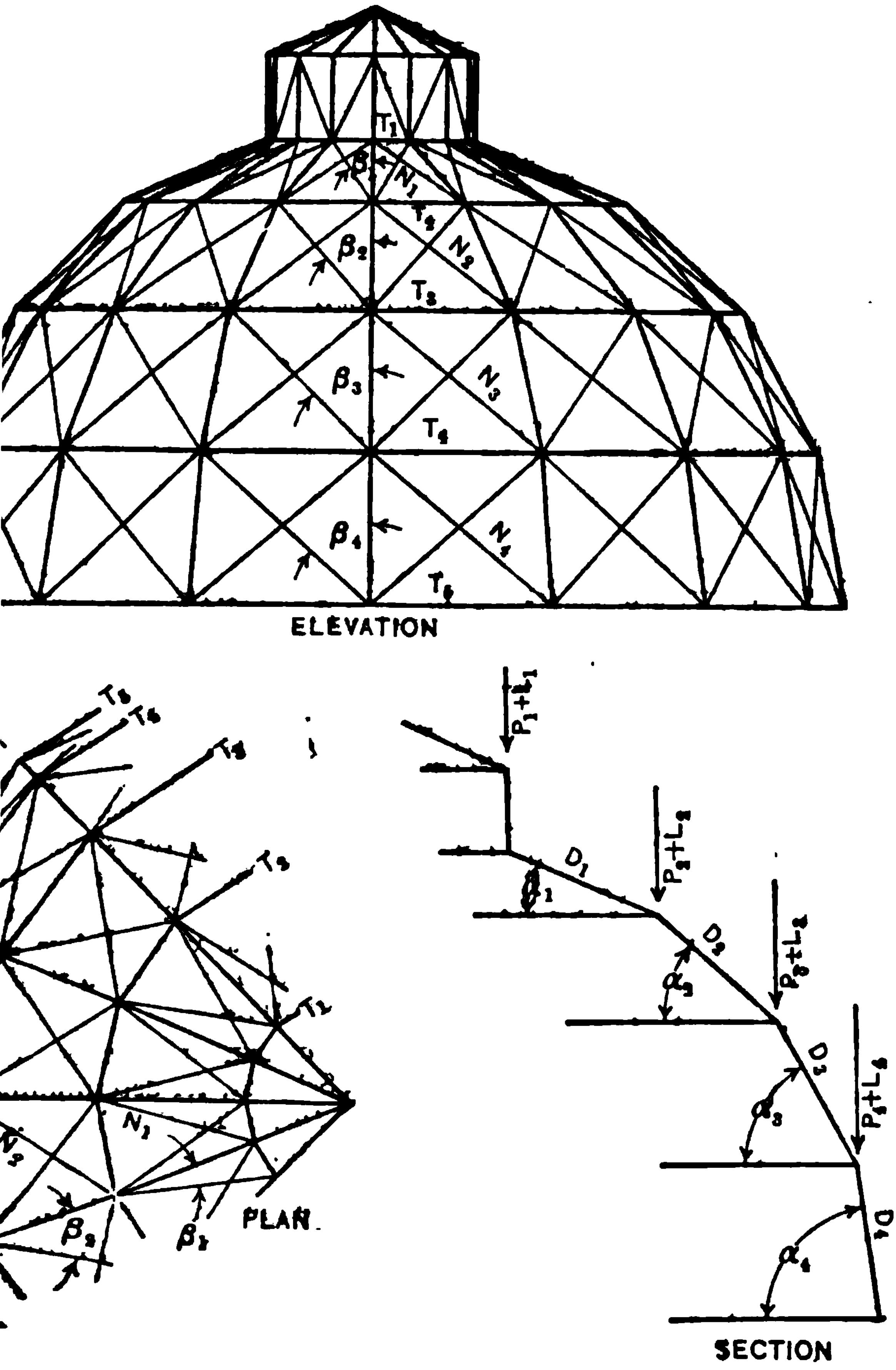


Fig. 4. Schwedler Ribbed Dome

$$D_1 = \frac{P_1 + L_1}{\sin \alpha_1} \quad D_2 = \frac{(P_1 + L_1) + (P_2 + L_2)}{\sin \alpha_2}$$

$$D_3 = \frac{(P_1 + L_1) + (P_2 + L_2) + (P_3 + L_3)}{\sin \alpha_3}, \text{ etc.}$$

$$T_1 = -\frac{(P_1 + L_1)\cot \alpha_1}{2 \sin \frac{\pi}{n}} = -\frac{D_1 \cos \alpha_1}{2 \sin \frac{\pi}{n}}$$

ult is negative the stress is compressive).

$$\text{Maximum } T_2 = \frac{(P_1 + L_1)\cot\alpha_1 - (P_1 + L_1 + P_2)\cot\alpha_2}{2\sin\frac{\pi}{n}}$$

$$\text{Minimum } T_2 = \frac{P_1\cot\alpha_1 - (P_1 + P_2 + L_2)\cot\alpha_2}{2\sin\frac{\pi}{n}}$$

$$\text{Maximum } T_3 = \frac{(P_1 + L_1 + P_2 + L_2)\cot\alpha_2 - (P_1 + L_1 + P_2 + L_2 + P_3)\cot}{2\sin\frac{\pi}{n}}$$

$$\text{Minimum } T_3 = \frac{(P_1 + P_2)\cot\alpha_2 - (P_1 + P_2 + P_3 + L_3)\cot\alpha_3}{2\sin\frac{\pi}{n}}, \text{ etc.}$$

$$N_1 = \frac{L_1}{2\sin\alpha_1\cos\beta_1} \qquad N_2 = \frac{L_1 + L_2}{2\sin\alpha_2\cos\beta_2} \qquad N_3 = \frac{L_1 + L_2 + L_3}{2\sin\alpha_3\cos\beta_3}, \text{ etc.}$$

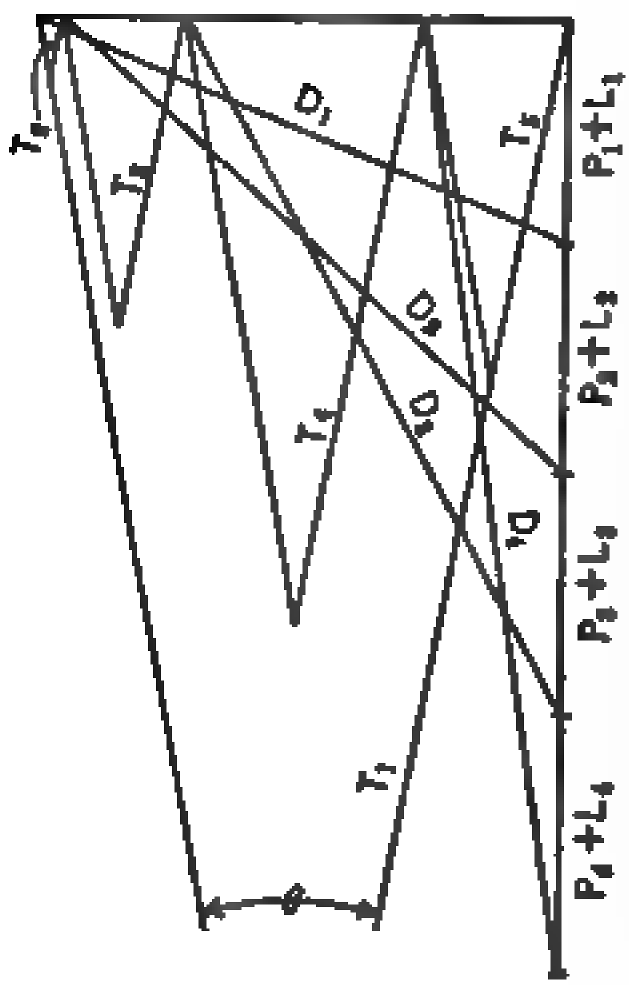


Fig. 4A. Graphical Determination of Stresses in Ribbed Domes

For the stresses in the diagonals factor 2 is introduced because Mü Breslau found, by exact anal stresses only one half as large as t determined by the simple resoluti forces. The diagonals are stressed u a wind-load, and this is resiste assuming a vertical live load equ from 20 to 30 lb per sq ft of HORIZO PROJECTION.

A GRAPHICAL METHOD, develope E. Schmidt, for determining the str D_1, D_2, D_3, T_1, T_2, T_3, etc., is show Fig. 4A.

Weights of Steel Domes. It found by Scharowsky, from calculat made for a large number of Schwe FLAT DOMES varying in span from 180 ft, that the weight of the la and steel skeleton per sq ft of proje (covered) area is

$$w = 0.0156S + 4$$

where w = pounds per square fo projected area, and S = the span, in For preliminary calculations on HEMISPHERICAL DOMES, the w found by this equation should b creased from two and a half to times.

Dome of the Horticulture Palace, San Francisco, Cal.* This is a LER HEMISPHERICAL DOME of 152-ft span, with twenty-four latticed ribs, :p, carrying a LANTERN-RING OR SPIDER at the top, and connected by eleven al rings. The lantern-ring is 6 ft in diameter, 36 in deep, with a solid l braced twice diametrically. The ribs are constructed of two 4 by 4 ι angles at the top, two 3 by 3 by 5⁄16-in angles at the bottom, and a 2½ ιy ¼-in angle single-lattice web. The dome-steel weighs about 17 lb : of projected area.

ete Ribbed Domes. In a REINFORCED-CONCRETE RIBBED DOME ber of ribs, varying from eight upward, is determined by the substruc- the size of the dome. The different steps in designing a ribbed rein- ncrete dome are: (1) the determination of the number of ribs and rings; etermination of the loading, per rib, using the required shell-thickness assumed rib-sizes and ring-sizes for preliminary calculations; (3) the f the forces acting on the ribs by the use of Schwedler's formulas; awing of the ELASTIC CURVE for the ribs; (5) the determination of the nd necessary reinforcement in the ribs, rings, and slabs; (6) the adjust- izes and loads, so as to be on the side of safety; and (7) the reworking eliminary computations for the final design. The ELASTIC CURVE vays remain in the MIDDLE HALF † OF THE RIB, and should never be y from the center of gravity of the rib-section that the maximum com- ress of 500 lb per sq in in the outer fiber of the rib is exceeded. The ient in the RIBS should be sufficient to resist the flexural stresses due ENTRICITY OF THE ELASTIC CURVE. The reinforcement in the RINGS sufficient to resist the tensile stresses, and should be as straight as ι order to avoid a sidewise stress or movement. The rings must be to resist their FLEXURE, as beams. The panel-slabs, if domical (see ιell Domes), should be reinforced for SHRINKAGE-STRESSES and TEM- STRESSES, in addition to the reinforcement for tension below the gle. If the slabs are straight they should be designed as floor-slabs, ilar methods.

.‡ It is required to build a dome (Fig. 5) with a span of 132 ft and a : 6 in. This makes the radius 85 ft. The eye is to be 12 ft in diam- outer surface of the dome is to be a domical slab on ribs, carrying a plastered ceiling forming the inner surface.

§ To obviate the necessity of building a complete domical form, rom the floor below, the decision is to build a RIBBED DOME as follows: d to build a central tower to temporarily carry the upper ends of the ast these ribs, raise them into position, cast the ring of the eye, sus- ng-forms from the ribs, pour the rings in place, and then fill in the on forms supported from the rings and ribs.

SPAN of the dome is 132 ft, the CIRCUMFERENCE at the base is 414.7 e of the suspension of the panel-forms, it is well to keep the PANEL-

rl and T. F. Chase, Engineering Record, Oct. 24, 1914.

l practice has been to keep the resistance-line within the MIDDLE THIRD in lly. In reinforced-concrete arches and domes it may depart a small dis- the middle third, but there should be sufficient steel to resist any tension The ribs of domes differ from ordinary arches as they are rigidly braced ιnd the slab-panels.

: of this example is similar to the dome over the Hippodrome at Copen- rk, by Christiani and Nielsen. See the periodical, Concrete, for December,

ution of this example all calculations have been made with the slide-rule.

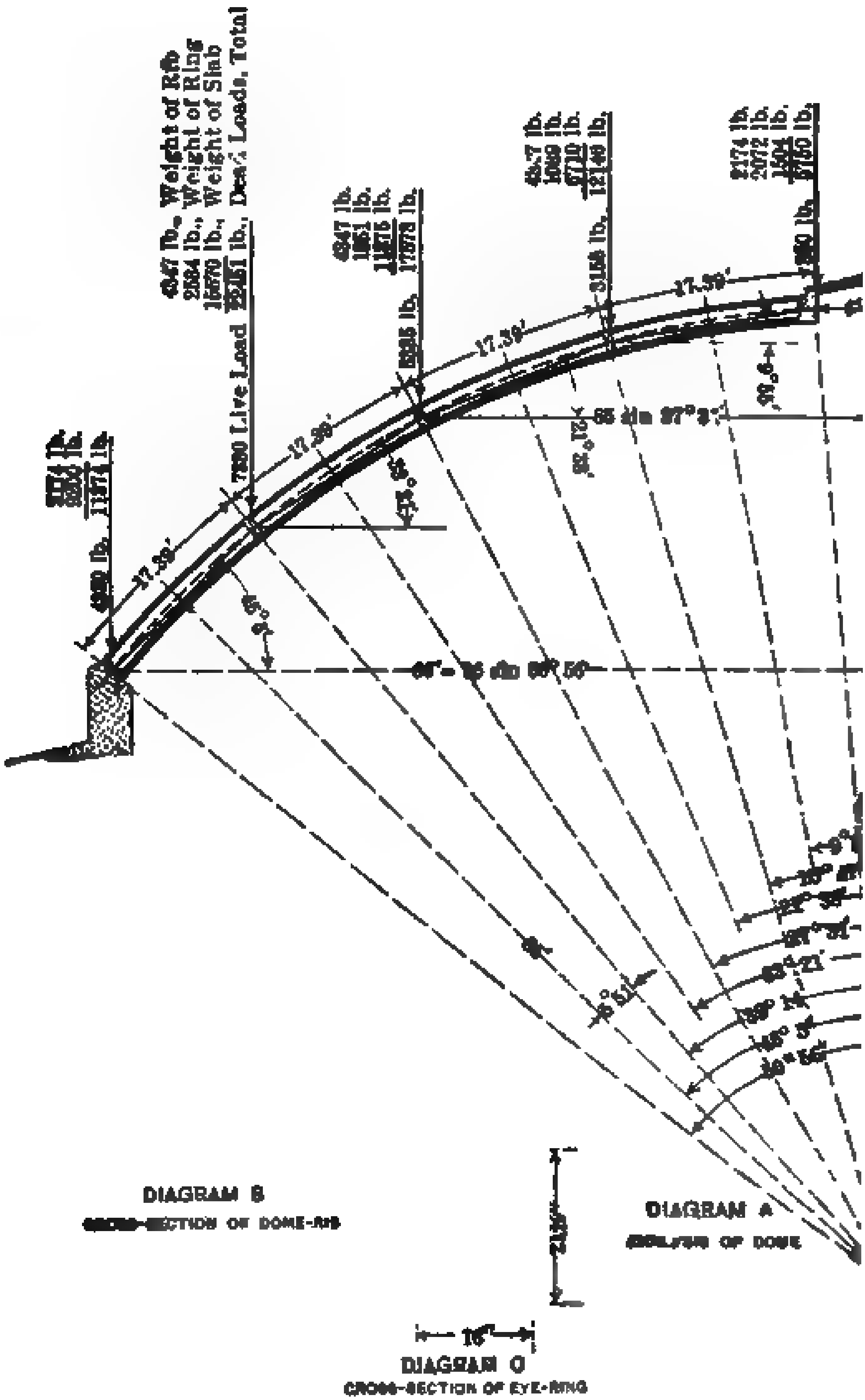


Fig. 5. Segmental Concrete Ribbed Dome

within about a 20-ft limit. Hence twenty ribs are necessary. With TERMEDIATE RINGS, the lower panels are approximately square. The : not to show below the ceiling, and hence a narrower spacing toward is unnecessary for appearance. For preliminary calculations, allowing per lin ft for the eye-ring and the load due to a glass covering over 250 lb per ft for the weight of the ribs, and 150 lb per ft for the weight termediate rings; and assuming a slab-thickness of 3½ in, a suspended eiling ¾ in thick, and 25 lb per sq ft of surface for snow-loads and wind- an equivalent of a 2-in thickness of concrete), the loading on the ribs is ι in Diagram *A* of Fig. 5. To illustrate the METHOD OF DETERMINING DS, the calculations for the loading at the lower intermediate ring are The WEIGHT OF THE RIB is $250 \times 17.39 = 4\,347$ lb. The WEIGHT OF G is $150 \dfrac{2 \times \pi \times 85 \times \sin 39° 14'}{20} = 2\,534$ lb. The WEIGHT OF THE D CEILING between $\theta = 33° 21'$ and $\theta = 45° 5'$ is, from the curves in with $\dfrac{cr}{a} = 0$,

$$\frac{\cdot\left(\frac{3\frac{1}{2}+\frac{3}{4}}{12}\right)(85)^2(1.84) - 150\left(\frac{3\frac{1}{2}+\frac{3}{4}}{12}\right)(85)^2(1.03)}{20} = 15\,570 \text{ lb}$$

l dead load is

$$4\,347 + 2\,534 + 15\,570 = 22\,451 \text{ lb}$$

:al live load is

$$\frac{15\,570 \times 2}{3\frac{1}{2} + \frac{3}{4}} = 7\,330 \text{ lb}$$

ess D_4 (see method in Fig. 4A), in the lower section of the rib is the nd according to Schwedler's formulas, page 1223, is

$$\frac{6\,600 + 15\,301 + 22\,908 + 29\,781}{\sin 45° 5'} = 105\,400 \text{ lb}$$

NTRICITY of the stress D_4 is

$$85 - (85 \cos 5° 51') = 0.445 \text{ ft}$$

ent due to the ECCENTRICITY of D_4 is

$$105\,400 \text{ lb} \times 0.445 \text{ ft} = 46\,903 \text{ ft-lb, or } 562\,836 \text{ in-lb}$$

st the COLUMN-LIKE COMPRESSION of D_4 there is required a cross-sec- ι of rib of

$$\frac{105\,400}{500} = 211 \text{ sq in of concrete}$$

st the effect of the ECCENTRICITY of the stress D_4, it is necessary to ugh steel so that the total stress in it, multiplied by the distance ie top and bottom steel reinforcements, is equal to the moment, 562 836 idy found.

ie ECCENTRICITY of D_4 is 0.445 ft, and the line of action of the thrust pt within the MIDDLE HALF OF THE RIB,* the rib will be

$$4 \times 0.445 = 1.78 \text{ ft} = 21\tfrac{1}{2} \text{ in in depth}$$

* See foot-note on page 1225.

Allowing 1½ in of concrete for steel-protection at the top and bottom of t rib, the distance between the inner and outer reinforcements is

$$21\tfrac{1}{2} - 3 = 18\tfrac{1}{2} \text{ in}$$

Therefore a stress of

$$\frac{562\,836}{18\tfrac{1}{2}} = 30\,424 \text{ lb}$$

is to be resisted by the steel at the top and bottom. Since there is steel in bo COMPRESSION and TENSION, the allowable UNIT STRESS in it is

$$\left(\frac{650 \times 18\tfrac{1}{2}}{21\tfrac{1}{2}}\right)(15 - 1) = 7\,830 \text{ lb per sq in}$$

This is because the allowable compressive unit stress in the outer fibers of co crete beams is 650 lb per sq in; the ratio of the MODULUS OF ELASTICITY of t steel and of concrete, 15; and the distance between the inner and outer st reinforcements, and the distance of the rib-depth, 18½ and 21½ in respectivel The 1 in the expression (15 − 1) is to take care of the stress carried by t concrete replaced by the steel.

The TOTAL CROSS-SECTION of steel necessary at both the top and bottom of t rib is, therefore,

$$\frac{30\,424}{7\,830} = 3.89 \text{ sq in}$$

furnished by four 1⅛-in round rods. The best arrangement of the 211 sq of concrete, and the steel, results in a cross-sectional shape shown in Diagram of Fig. 5. The STIRRUPS should be spaced not more than three fourths of th distance between lines of longitudinal steel, or ¾ × 18½ = 12 in (approximatel and they should be made from ⅜-in round rods. Because of the TIES in th flanges, it is advisable to use small ¼-in rods as STIFFENERS at the inte sections of the ties and stirrups. Projecting LOOPS should be left for fastenin the panel-slabs. The actual weight per linear foot of the ribs is

$$\frac{211}{144} \times 150 = 220 \text{ lb}$$

for the concrete, plus

$$(4 + 3.38) + 11 = 25 \text{ lb}$$

for the steel, equal to a total of 245 lb, as against 250 lb per lin ft previousl allowed in the calculations.

As the ribs are to be precast and raised into place, it is necessary to determin whether they are of sufficient strength for this, and whether they will stand, u supported by the rings, without breaking under their own weight. By conside ing the ribs to be simple arches, and testing them by determining the line thrust, it is found that they are amply safe. In order to resist the thrusts stresses developed by raising the ribs into place, it is necessary to tie the en together with bow-string rods.

The stresses in all of the rings, except the FOOTING-RING, are compressiv This is because they are all above the CRITICAL ANGLE. (See Smooth-She Domes.) Therefore, in determining the stresses by Schwedler's formulas, onl the equations for the MINIMUM VALUES of T_1, T_2, T_3, and T_4 need be used.

The stress in the EYE-RING is

$$-\frac{(5\,750 + 850)\cot 9^\circ\,55'}{2\sin\frac{\pi}{20}} = -120\,600 \text{ lb}$$

ress in the FIRST INTERMEDIATE RING is

$$\frac{750 \cot 9° 55') - (5\,750 + 12\,146 + 3\,155) \cot 21° 38'}{2 \sin \frac{\pi}{20}} = -65\,000 \text{ lb}$$

ess in the SECOND INTERMEDIATE RING is

$$\frac{+ 12\,146) \cot 21° 38' - (5\,750 + 12\,146 + 17\,573 + 5\,335) \cot 33° 21'}{2 \sin \frac{\pi}{20}}$$

$$= -53\,900 \text{ lb}$$

ess in the THIRD INTERMEDIATE RING IS

$$\frac{12\,146+17\,573) \cot 33° 21' - (5\,750+12\,146+17\,573+22\,451+7\,330) \cot 45° 5'}{2 \sin \frac{\pi}{20}}$$

$$= -35\,600 \text{ lb}$$

ess in the FOOTING-RING is tensile, and hence the equation for the MAX-
LUE of T_5 gives

$$\frac{+ 850 + 12\,146 + 3\,155 + 17\,573 + 5\,335 + 22\,451 + 7\,330) \cot 45° 5'}{2 \sin \frac{\pi}{20}} = 238\,000 \text{ lb}$$

-RING should have $\frac{120\,600}{500} = 242$ sq in of concrete, but for appearance it e as wide as or wider than the ribs; hence it is made $21\frac{1}{8}$ in high and 16 This size allows, also, a firm ANCHORAGE for the rib-reinforcing. With einforcing it requires four $1\frac{1}{8}$-in round rods. (See Diagram C, Fig. 5.) intermediate ring should be

$$\frac{65\,000}{500} = 130 \text{ sq in}$$

ection, requiring a 7-in width and a $18\frac{1}{2}$-in height, to resist the load, as a As the ring must also act as a BEAM, carrying its own weight, the f half the slab, and the live load (the forms taking the place of the live ng construction), steel must be added to resist the BENDING MOMENT

$$\frac{\left(\frac{130}{144} \times 150\right) \left(\frac{2\pi 85 \sin 15° 47'}{20}\right)^2 + \left(\frac{6\,710 + 3\,155}{2}\right) \left(\frac{2\pi 85 \sin 15° 47'}{20}\right)}{12}$$

70 ft-lb = 42 840 in-lb

steel, in TENSION and COMPRESSION, must be added to keep the addi- ess in the concrete, due to this moment, down to 150 lb per sq in, since = 650 lb per sq in is the MAXIMUM ALLOWABLE COMPRESSIVE STRESS ncrete of the beam. From Formula (1) page 925, or Formula (5) page

$$K = \frac{42\,840}{7 \times (17)^2} = 21.2$$

mulas (2), (3) and (4), (pages 925–6), when $K = 21.2$ and $S_t = 16\,000$ in; $S_c = 245$ lb per sq in, $p = 0.0014$, and $x = 0.185$. Since S_c must

not exceed 150 lb per sq in, it is necessary to add COMPRESSION-STEEL to resist stress of

$$\left(\frac{245-150}{2}\right)(7)(0.185\times 17) = 1\,040 \text{ lb}$$

The allowable stress in the COMPRESSION-STEEL (page 1228), less the stress already allowed for the concrete which is replaced by the steel, if placed 1½ in from the outside, is

$$\left(\frac{650}{0.185\times 17}\right)\left((0.185\times 17)-1.5\right)(15-1) = 4\,770 \text{ lb per sq in}$$

The amount of COMPRESSION-STEEL is, therefore,

$$\frac{1\,040}{4\,770} = 0.22 \text{ sq in, cross-section}$$

The tensile-steel necessary is

$$0.0014\times 7\times 17 = 0.16 \text{ sq in}$$

but because of the NEGATIVE MOMENT at the ribs, the same cross-sectional area is used as for COMPRESSION, that is, 0.22 sq in, furnished by two ⅜-in round rods. The UNIT-SHEAR is

$$\frac{\left(\frac{130}{144}\times 150\right)\left(\frac{2\pi 85 \sin 15^\circ 47'}{20}\right)+\left(\frac{6\,710+3\,155}{2}\right)}{7\times 17\times\left(1-\frac{0.185}{3}\right)\times 2} = 18.6 \text{ lb per sq in}$$

No STIRRUPS are necessary to resist shear, but stirrups made from ¼-in round rods should be spaced about 18 in on centers, to tie the panel-slabs securely to the ring.

The SECOND INTERMEDIATE RING, if made the same size as the first, will have a stress of

$$\frac{53\,900}{7\times 18\tfrac{1}{2}} = 416 \text{ lb per sq in}$$

The MOMENT will be

$$\frac{\left(\frac{130}{144}\times 150\right)\left(\frac{2\pi 85 \sin 27^\circ 31'}{20}\right)^2+\left(\frac{11\,375+5\,335}{2}\right)\left(\frac{2\pi 85 \sin 27^\circ 31'}{20}\right)}{12}$$

$$= 10\,300 \text{ ft-lb} = 123\,600 \text{ in-lb}$$

From Formulas (1), (2), (3), and (4), (pages 925–6), $K = 61.2$, $S_c = 455$ lb per sq in, $p = 0.0043$, and $x = 0.3$. Since S_c cannot exceed $650 - 416 = 234$ lb per sq in, the COMPRESSION-STEEL must resist

$$\left(\frac{455-284}{2}\right)(7)(0.3\times 17) = 3\,930 \text{ lb}$$

The section-area of the COMPRESSION-STEEL is, therefore,

$$\frac{3\,930}{\left(\frac{650}{0.3\times 17}\right)\left((0.3\times 17)-1.5\right)(15-1)} = 0.62 \text{ sq in}$$

furnished by two ¾-in round rods at top and bottom. The UNIT SHEAR is

$$\frac{\times 150)\left(\frac{2785 \sin 27^\circ 31'}{20}\right)^2 + \left(\frac{11\,375 + 5\,335}{2}\right)}{7 \times 17 \times \left(1 - \frac{0.3}{3}\right) \times 2} = 46.9 \text{ in per sq in}$$

efore necessary to resist 46.9 − 40 = 6.9 lb per sq in of shear, with that is, with two ¼-in round-rod stirrups, spaced 12 in apart at the the others 18 in apart through the remaining distances.

IRD INTERMEDIATE RING is 7 by 18½ in in section, with two ⅞-in s at top and bottom, and with two ⅜-in round-rod STIRRUPS, spaced at the ends, two more, spaced 12 in, and the rest spaced 18 in.

MENT due to the ECCENTRICITY of the COLUMN-LIKE THRUST, that is, udinal horizontal compressive stress, in the rings is resisted by the slabs. act analysis may be made by considering only the NORMAL COMPONENTS ADS on the rings in determining these moments.

TING-RING must have enough tensile-steel to resist the outward PUSH of the ribs, that is $\frac{238\,000}{16\,000} = 14.9$ sq in of steel cross-section. In this, if the ring acts as a BEAM, there must be sufficient steel to resist it due to the combined weights of the dome and the ring itself.

EL-SLABS being domical and above the critical angle, are in com- nd should be designed as illustrated in the discussion of Smooth- es.

2. Vaults *

ation. Vaults may be conveniently considered under the following (1) Barrel vaults, (2) Groined vaults, and (3) Ribbed vaults (Masonry, amed).

Considerations. A knowledge of the ELASTIC THEORY OF ARCHES bility of buttresses is necessary in a rigid investigation of vaults, since involves the application of the principles of that theory. (See, also, II and VIII.) In any vault, lines of action of the stresses or thrusts through the material between certain limiting lines; otherwise the ail. These thrusts are brought to the grade-line, or to foundations, ften buttressed in the case of barrel vaults, and by piers and but- he case of groined and ribbed vaults. By building vaults of light uch as hollow bricks or hollow tiles, the magnitude of the thrusts are nd lighter walls, piers, or buttresses can be used.

Fig. 1. Three Methods of Building Barrel Vaults

aults. Fig. 1, (*a*), (*b*), and (*c*), illustrates three methods of building LTS. In (*c*) the longitudinal ribs are merely for appearance, as

* atment of this subject may be found in the Handbuch der Architektur, 's Baukonstructions Lehre.

they do not strengthen the vault. The diagrams (*a*) and (*b*), Fig. 2, illustrat methods of disengaging the masonry of barrel vaults from the walls. Dia (*b*) is the better me and improves the ap ance of the vault o inside. Diagrams (*c*) (*d*) illustrate the u stone skewbacks for mental vaults.

(*a*) (*c*) (*d*) (*b*)

Fig. 2. Methods of Joining Barrel Vaults to Walls

Strength of B Vaults. Barrel vaults be considered as a se OF ARCHES set next to other; and hence if a tion one unit long is fo safe when investigated an arch, the vault itse considered safe. By be ing the wall and the v together as a unit, point on the arc 60° from the vertical or crown, that is, to a point on the trados one third of the distance from the horizontal spring-line, the actual is materially decreased. With the spring-line at 60°, the line of thrust in unloaded arch or barrel vault of an equal thickness throughout, will remain within a strip whose radial thickness or width is about one forty-second of the radius. If the line of thrust is to remain within the middle third of the arch-ring or vault-ring, t should be $(r/42) \times 3 = r/14$. If it is to remain within the middle half, t should be $(r/42) \times 2 = r/21$. In the following example, the theory of the middle half will be followed, in which $t = r/21$. If it were assumed that $t = r/14$, the line of thrust being kept within the middle third, the span of the vault in the example would have to be changed from 21 to 14 ft.

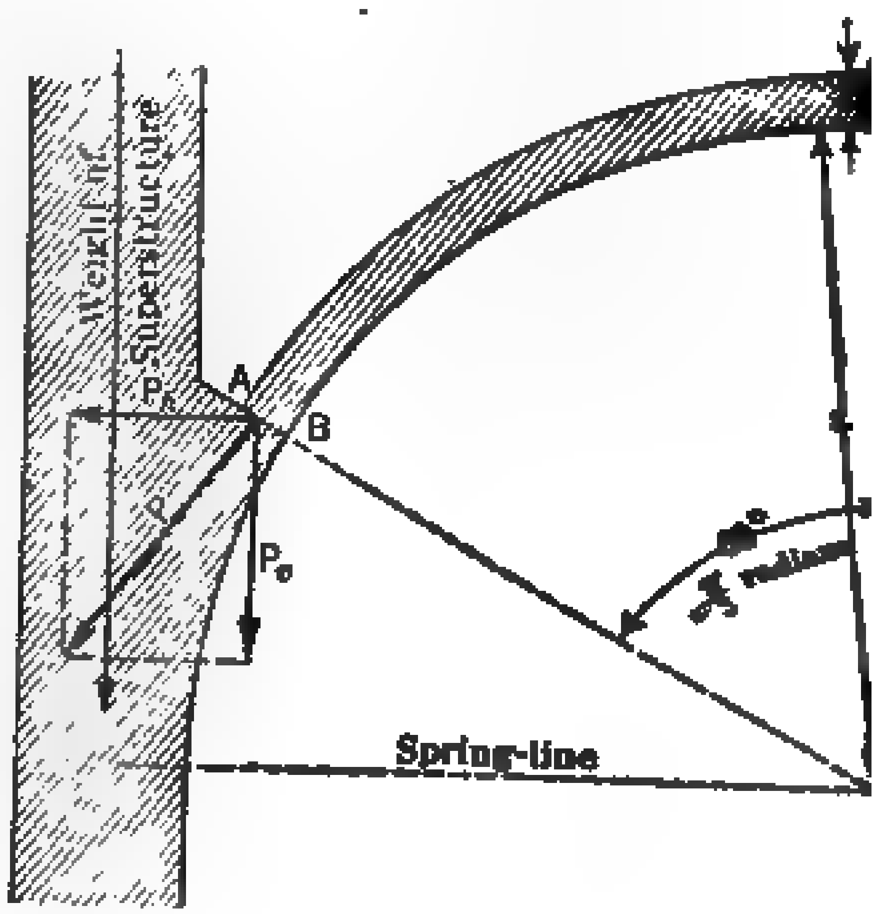


Fig. 3. Analysis of Barrel Vault

If built, then, as described (Fig. 3), the minimum thickness of the unloa vault-shell is about one twenty-first of the vault-radius, that is,

$$t = r/21$$

The VERTICAL COMPONENT P_v of the thrust P is equal to the weight of half free vault, that is, of the section $ABCD$. It can be shown that the HORIZON

ENT P_h of the thrust is 0.79 of the vertical component, and that the s not at right-angles with the spring-line AB; that is,

$$P_h = 0.79P$$

ple. It is required to construct a BARREL VAULT over a corridor 21 ft The vault-radius is 10½ ft, and the minimum thickness of the shell is $= 0.5$ ft $= 6$ in. If built of bricks it is cheaper to build a ribbed vault, as -dimensions of bricks are approximately 4 in, 8 in, 12 in, etc. Referring ., it is found that a 4-in vault with ribs 4 by 8 in every 3 ft 3 in, is equiva-

6. Barrel Vaults, Ribbed and Non-ribbed. Equivalent Thicknesses

6-in vault, and hence would be used. The brick masonry weighs cu ft.

RTICAL COMPONENT P_v of the thrust is

$$\frac{[\tfrac{1}{4} \times 1/3\pi \times 10.5 \times 125] + [4/12 \times 8/12 \times 1/3\pi(10.5 + 0.33) \times 125]}{3\tfrac{1}{4}}$$

$$= 557 \text{ lb per lin ft}$$

ZONTAL COMPONENT P_h of the thrust is $0.79 \times 557 = 440$ lb per lin ft. rting wall must be thick enough, buttressed enough, or loaded suf-rom above, to take care of this horizontal component of the thrust. s a graphical analysis of the stresses in this vault It will be noticed hat the line of pressure remains in the MIDDLE HALF of the vault-thick-effler, after numerous tests of vaults, stated * that if one fourth the

* Theorie der Gewölbe.

vault-thickness is deducted at the extrados, and one fourth at the intrados, an that if the line of pressure found according to the elastic theory of arches

Fig. 5. Graphical Determination of Stresses in a Barrel Vault

confined to the remaining portion, that is, the MIDDLE HALF, then the vault m be considered safe. Fig. 6 shows the resistance-line passing slightly outside t

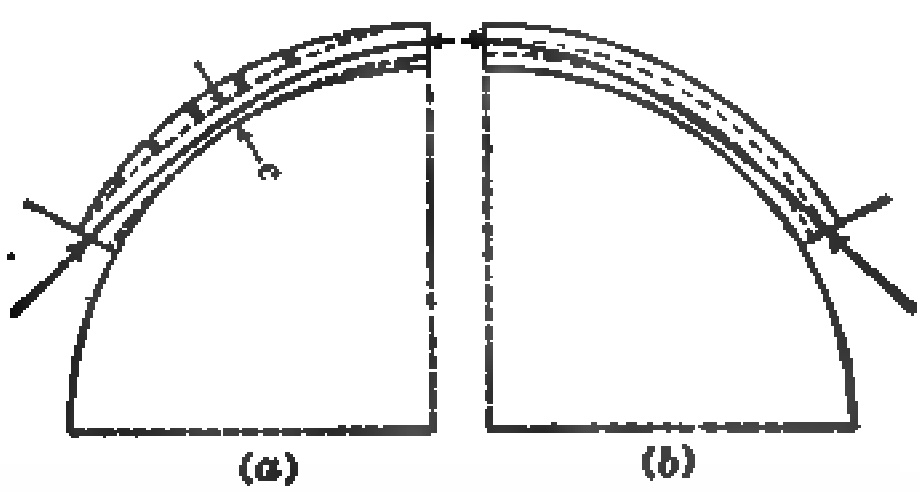


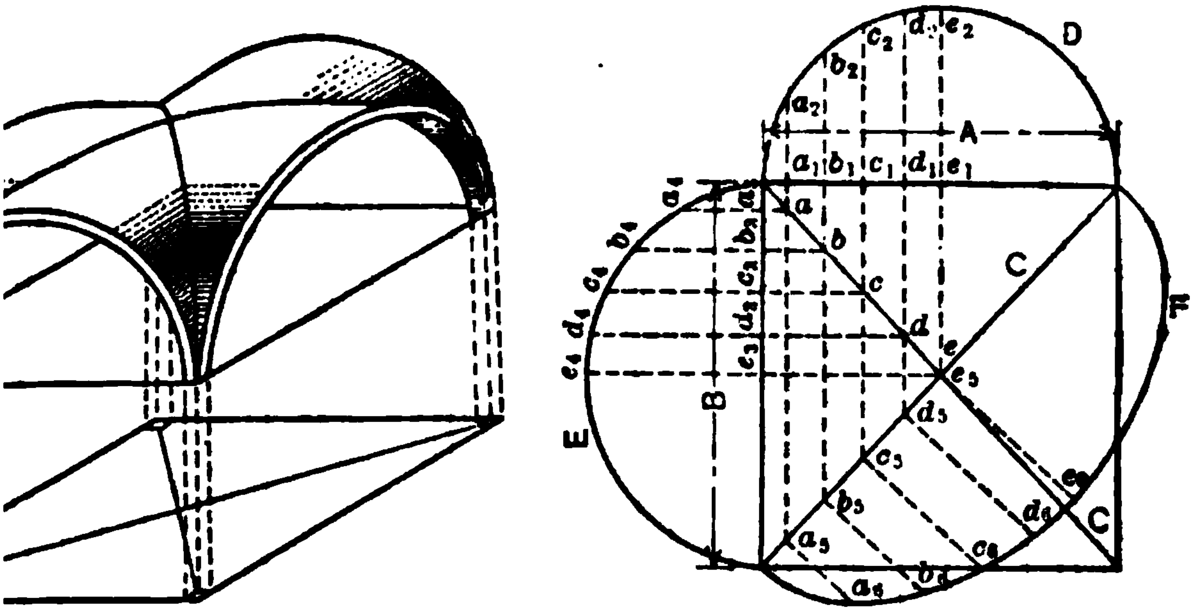
Fig. 6. Line of Pressure through Vault-thickness

middle third.* It illustrates the less conservative theory that the resistan line might in some cases pass near the outside of the middle half. The arch vault in Diagram (b) of Fig. 6 would have a greater tendency to fail accordi to the middle-third theory, because the line of pressure or resistance-li passes outside of the middle third. Diagram (a) of Fig. 6 shows the same a or vault with the shell cut so that the line of pressure passes down the exact cen of the uncut portion. This results in a sort of theoretical or ideal arch-for

* See foot-note relating to Concrete Ribbed Domes, page 1225.

se the thickness of part c must be sufficient to develop a safe compres-
istance in the material, and it is advisable to add sufficient steel to take
ny tension in the parts farthest from the resistance-line. Vaulted con-
n is often relatively protected and free from the live loads and moving
o which arches are generally subjected; and for such construction
·'s conclusions are considered valid.

ied Vaults. A GROINED VAULT is formed by the intersection of two
VAULTS. (See (a), Fig. 7.) By using groined vaults it is possible to



(a) ective, Showing Penetrations and Intersections

(b) Intersecting Vaults of Different Widths

Fig. 7. Groined Vault

tops of windows and doors above the spring-lines of the vaults, and to
te the pressures or thrusts on piers or columns.

. The intersections of two vaults, called GROINS, are straight lines in
projection, only when they are of the same curvature and height.
lts are of different widths, it is best to make one semicircular, draw
ntal projections of the groins as straight lines, and then determine the
the other vault. This is illustrated in Fig. 7 (b). Vault A is semi-
d has a span A. Vault B has a span B. CC are the GROINS, and D is
r contour of the narrow vault. Any points, a, b, c, etc., are chosen at
d lines a–a_1, b–b_1, c–c_1, etc., and a–a_4, b–b_4, c–c_4, etc., drawn parallel to
the respective vaults. The line a_3–a_4 is laid off equal to a_1–a_2; b_3–b_4,
b_2, etc. The smooth curve connecting a_4, b_4, c_4, etc., is the contour E
t B. In like manner the contour F of the groins is found by similarly
$_5$–a_6, b_5–b_6, etc., equal to a_1–a_2, b_1–b_2, etc.

LT-SHELLS, at the intersections or groins, should never have what
lled MITER-JOINTS. The vaults should be monolithic or there should
d ribs to carry the vault-shells and transmit the thrusts to the piers.
secting vaults are of stone, and of the same diameter, the groins may
shown in Fig. 8 (a) for small vaults, or as in Fig. 8 (b) for larger
Fig. 8 (a) the GROIN-STONES are L-shaped and are cut so as to carry
ourses of one vault around to the other vault. The stone shown in
) of Fig. 8 is shown in plan at b, with two views at c and d. A better
hown in Fig. 8 (b). Here the groin-stones are cut so that the joints

are normal to the groins, thus forming concealed ribs. This bearing-surfa is obtained as follows. Point *a*, the intersection of an extended vault-joint a the groin-edge, is projected down to *a′* and *b′*, the intersections of the projecti line and the assumed side and center lines of the rib. Point *b′* is projected up

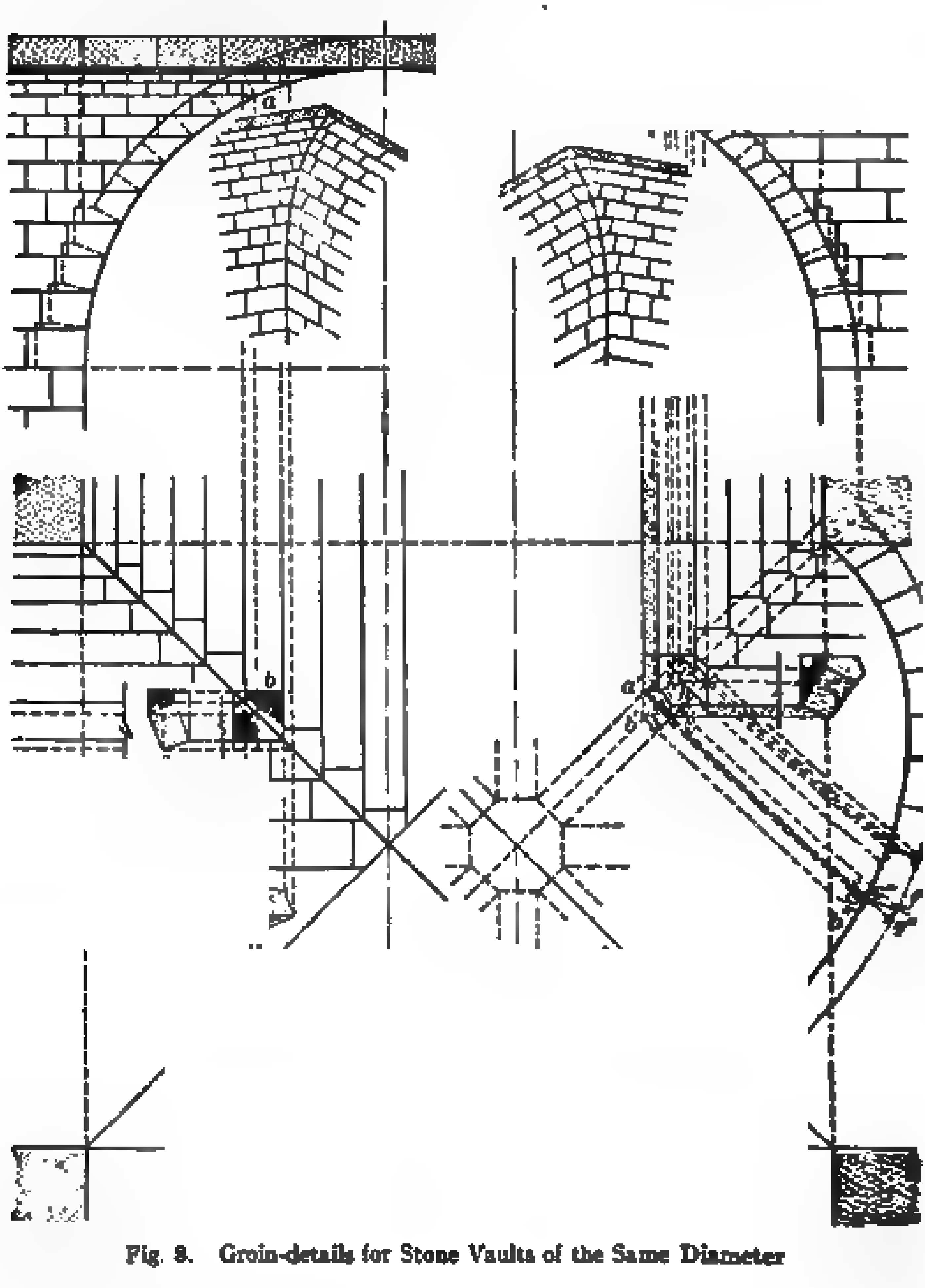


Fig. 8. Groin-details for Stone Vaults of the Same Diameter

b″, a point on the center line or edge of the rib. From *b″* a horizontal line int sects with a line projected up from *a′* to give *c″* a point on the joint, which drawn normal to the groin. The intersection *d″* of this joint with the gro edge is projected down to *d′* on the center line of the rib. By connect

ʹ, and d'' and e' (the point opposite a' on the other side of the rib)
line, the lower edge of the bearing-surface is determined.
d'' and e' projected up determine d and e, the same points in elev

g. 9. Groin-details for Stone Vaults of Different Diameters

s on these curved lines can be found by choosing points betw
rojecting them in the same way as in the method used to fi
of f. The procedure is as follows. The point f is projected

to the center line of the rib, locating g'. Then g' is projected up to g'', the i section with the line representing the joining of the upper surfaces of the va A horizontal line is projected over from g'' to f''', the point of intersection with normal joint. The point f''' is projected down to f', on the projecting line fro By connecting f' and h' (h' is opposite f' and equidistant from the center lin the rib) with a straight line, the upper edge of the bearing-surface is determi The point h is found by projecting up from h'. By connecting a' and f', a and h'; also a and f, and e and h, the side edges of the bearing-joint are loc The lower bearing-surface of a stone, or the upper bearing-surface of the lower stone, is found in a similar manner.

If the vaults are not of the same diameter, either of two methods may be u The number of stone courses in both vaults may be made the same, thus mak the courses in the wide vault wider than those in the narrow vault, and method of finding the shape of the groin-stones is similar to that shown in Fig. 8 or the stones may be the same width, thus making a greater number of cours the wide vault than in the narrow vault. In the latter case the groin-stones are termined as in Fig. 9. To take care of the different number of courses in the vaults, one course in the narrow vault is sometimes made to receive two co in the wide vault, as shown by stone A in Fig. 9. Because the joint a is hi than the joint b, there results a peak toward the side of the groin-line. Th cut off at right-angles to the groin, thus making the bearing-surface c. surface is determined as follows. The intersection of the joint-planes d a is at f. The vertical projection f_1, of f, is drawn through h_1, found by proje up h and g, and a horizontal line from g_1. The intersection of f_1 and a through g_1, normal to the groin-curve gives i_1, which, projected to i, gives intersection of the sides of the bearing-surfac

The point j is found by projecting up k, intersection of a and the diagonal, to k_1; projecting k_1 to j_1, the intersection with normal line; and then projecting j_1 to j. connecting g and j with a curved line (o points of which are determined by draw lines parallel to a and proceeding by the met used in finding j); and g and i, and j and i, straight lines; the sides of c are determined.

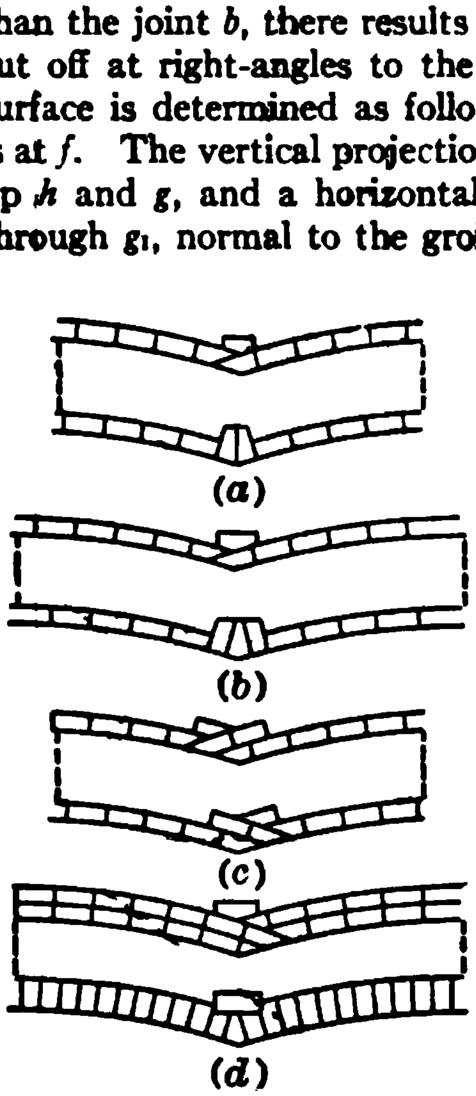


Fig. 10. Groins of Brick Vaults

If the vaults are built of brick, it is bette run the courses at right-angles to the gr thus giving a chance for the bricks to over as shown in Fig. 10. If the brick courses to run parallel to the center line of the vault is necessary to use stone ribs to carry the sh

Determination of the Stresses in Groi Vaults. The problem of a groined vault s ning a RECTANGULAR AREA which is not squ is here considered, as a vault spannin SQUARE AREA offers fewer difficulties and ca worked out on the same principles. The problem is to span an area, wh half-length of the short diameter is a, and whose half-length of the long diam is b, Fig. 11 (a). In order to obtain a more stable construction, the point of int section of the crowns of the vault is raised a distance $cd = c'd$, thus giving crown of the long-span vault a slope ce and the crown of the short-span v a slope $c'f$. The vault is divided into strips A, B, C, etc., and A', B', C', from the rib R, as shown in the projected area in Fig. 11 (a). The rib R

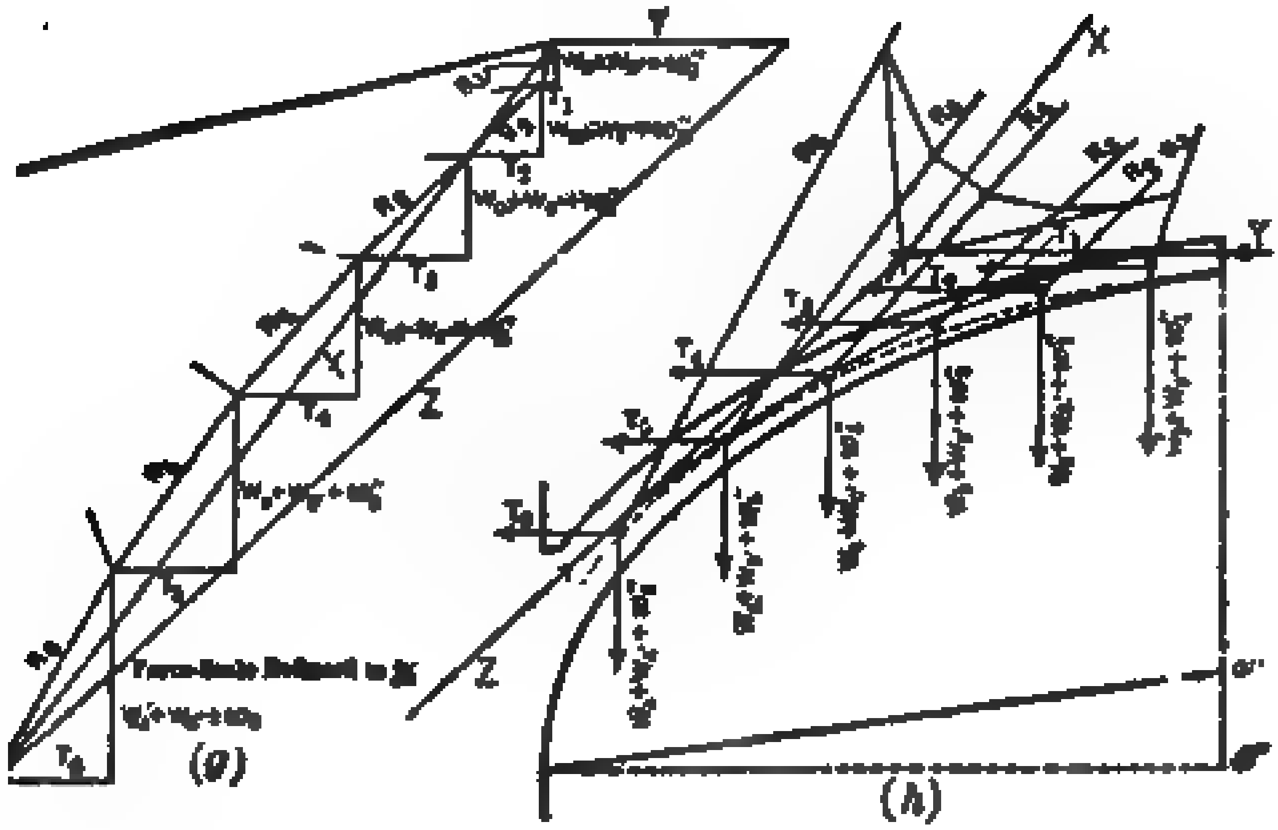


Fig. 11. Determination of Stresses in Groined Vaults

given a width equal to the assumed width of the supporting diagonal concea arch, and the widths A, B, C, etc., and A', B', C', etc., are obtained by dividi the two vaults into the same number of equal parts. These strips are conside as adjacent arches resting on the rib R. For simplification the line of pres or resistance-line of each strip is placed in the center of that strip as gk in and $g'k'$ in A'. The error in this is on the side of safety.

Even though the projected areas of the two intersecting vaults are the sa the actual surface-area of the smaller-span vault is slightly larger than that the longer-span vault. Therefore, if the vaults are of the same thickness, shorter-span vault is slightly heavier than the larger-span vault. In order have the resultants of the horizontal components of the thrusts from strip and strip A', strip B and strip B', etc., parallel to the direction of the rib the procedure is as follows.

The thrusts of the strips on the heavier side, that is of strips A, B, C, D, and F, are determined as shown in Fig. 11 (*b*) and (*c*). The curvature of strips being the same, the work can be considerably lessened by dividing arch into sections of unequal lengths for weight-determinations. The divid line for the sections is found, by projecting up the point of intersection of the of pressure of each strip and the side of the rib R, as g to g', h to h', etc. weights w_1, w_2, w_3, etc., of each section are then determined and the compo load-line drawn as in Fig. 11 (*c*). The positions of W_A, W_B, W_C, etc., Diagram (*b*) are determined by the usual STRESS-POLYGON. H is then drawn as to be at the upper limit, and the different thrusts so as to act near the lo limit of the middle half of the vault-thickness.* Lines drawn in Fig. 11 parallel to these thrusts, determine their values, and the values of the ho zontal components H_A, H_B, H_C, etc. The weights w'_1, w'_2, w'_3, etc., in Diagr (*d*), are found in the same way, the load-line in Diagram (*f*) drawn, and positions of W_A', W_B', W_C', etc., found as before. H' in Fig. 11 (*d*) is dra at the upper limit of the middle half in this demonstration.* H_A', H_B', H etc., however, must have such values that the resultants of H_A, and H_A', and H_B', etc., are parallel to R. The required values of H_A', H_B', H_C', e are found as in Fig. 11 (*e*), by laying off H_A, H_B, H_C, etc., and draw T_1, T_2, T_3, etc., parallel to R. The resulting values of H_A', H_B', H_C', e are then laid off in Fig. 11 (*f*) and the thrusts drawn. When drawing thrusts in Fig. 11 (*d*) through the intersection, of H' and W_A', H' and W etc., parallel to their directions in Fig. 11 (*f*), it is found that they act w slightly above the lower edge of the middle half.

The rib R is then drawn as in Fig. 11 (*h*) and the points of application of loads located. The LOAD-POLYGON is drawn as in Fig. 11 (*g*). The RESULTA R_1, R_2, etc., are drawn in both Diagrams (*h*) and (*g*) of Fig.11, the position of in Diagram (*h*) found by the usual STRESS-POLYGON, and the THRUSTS Z and determined. The point through which Z, Diagram (*h*), passes at the spring the rib, should be so chosen that the LINE OF PRESSURE remains at least wit the middle half of the rib; or the more usual and conservative limits of middle third may be used. In the case of brick vaults the strips A, B, etc., are taken at right-angles to the groin, resulting in vertical loads, only, the assumed rib.

Ribbed Vaults. In RIBBED VAULTS the ribs are designed to be built fi to be free-standing, and of sufficient strength to support the shell when placed over them. To simplify the construction, all the rib-arcs are ordina made with the same radius, thus making all the ribs disengage each other at

* The theory of the middle third is the one usually followed, as it is the most conser-tive and results in a larger factor of safety. See, also, foot-note on page 1225.

ight. This makes the narrower rib-arches pointed, and the diagonal
s semicircular, but they are all constructed of similar stones with cross-
of the same shape. To determine the points A and B (Fig. 12), at which
become independent of each other and of the wall, the proceeding is as
In plan the clustered ribs are shown just above the column-capitals,
diagonal ribs extending into the wall a distance ab. To find the height
w an arc through a with the same curvature as that of the diagonal rib,
at right-angles to the ribs, in plan, a line from b, until it cuts this arc at
eight cb is the height at A. The height at B, equal to fe, is found in the
r. The WEBS, or parts of the vault-shell supported by the ribs, are

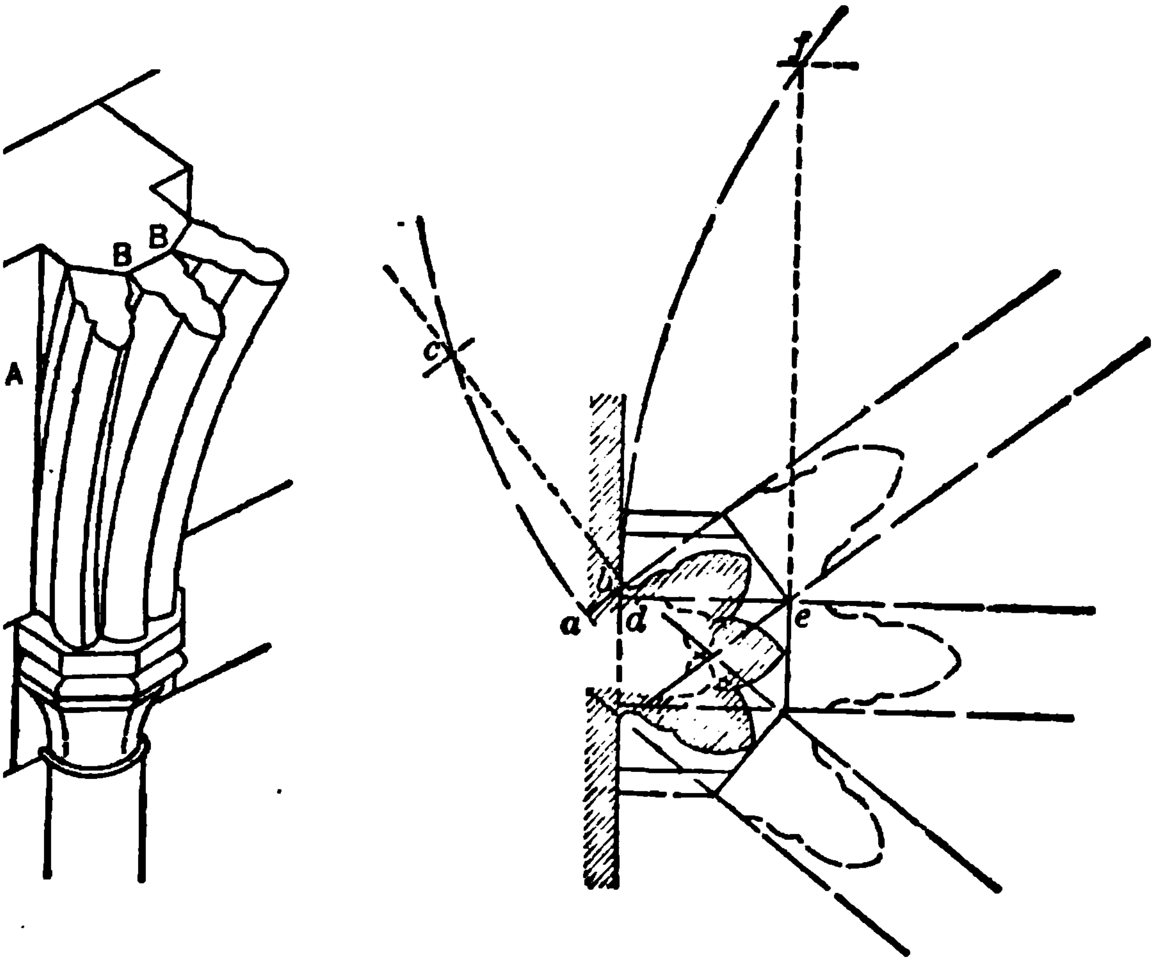


Fig. 12. Vault-rib Construction

llow arches in cross-section, and are SPHERICAL TRIANGLES, that is,
MICAL.

to use to the fullest advantage the finished lower portions of the vault
for the upper courses as laid, the courses of the VAULT-SHELL, or WEB
planes normal to the wall and the transverse ribs. This is shown in
projection in Fig. 13 I. The web being arched in both directions, the
in two directions, as in domes. From the study of the theory of
found that the THICKNESS OF THE SHELL in a dome has NO EFFECT ON
TY. The web in ribbed vaults being domical, can be made relatively
r stone or brick vaults it should not be less than about 4 in thick
to 35 ft.

are designed as arches, loaded with the thrusts of the web supported.
sts are determined as illustrated in Fig. 13 II. The vaulting
the half-wall, or transverse rib A, and the half-diagonal rib B, is
any number of equal LUNES, or figures bounded by the two inter-
, and radiating from the AXIS OF THE DOME of which that part of the
a SPHERICAL TRIANGLE. This axis is found by projecting, at right-
the ribs A and B, lines starting at the center of curvature of the ribs

and intersecting at the point e, which is the projection of the axis of the do
The RADIUS OF THE DOME is then R_1 in Fig. 13 II, equal to the distance from

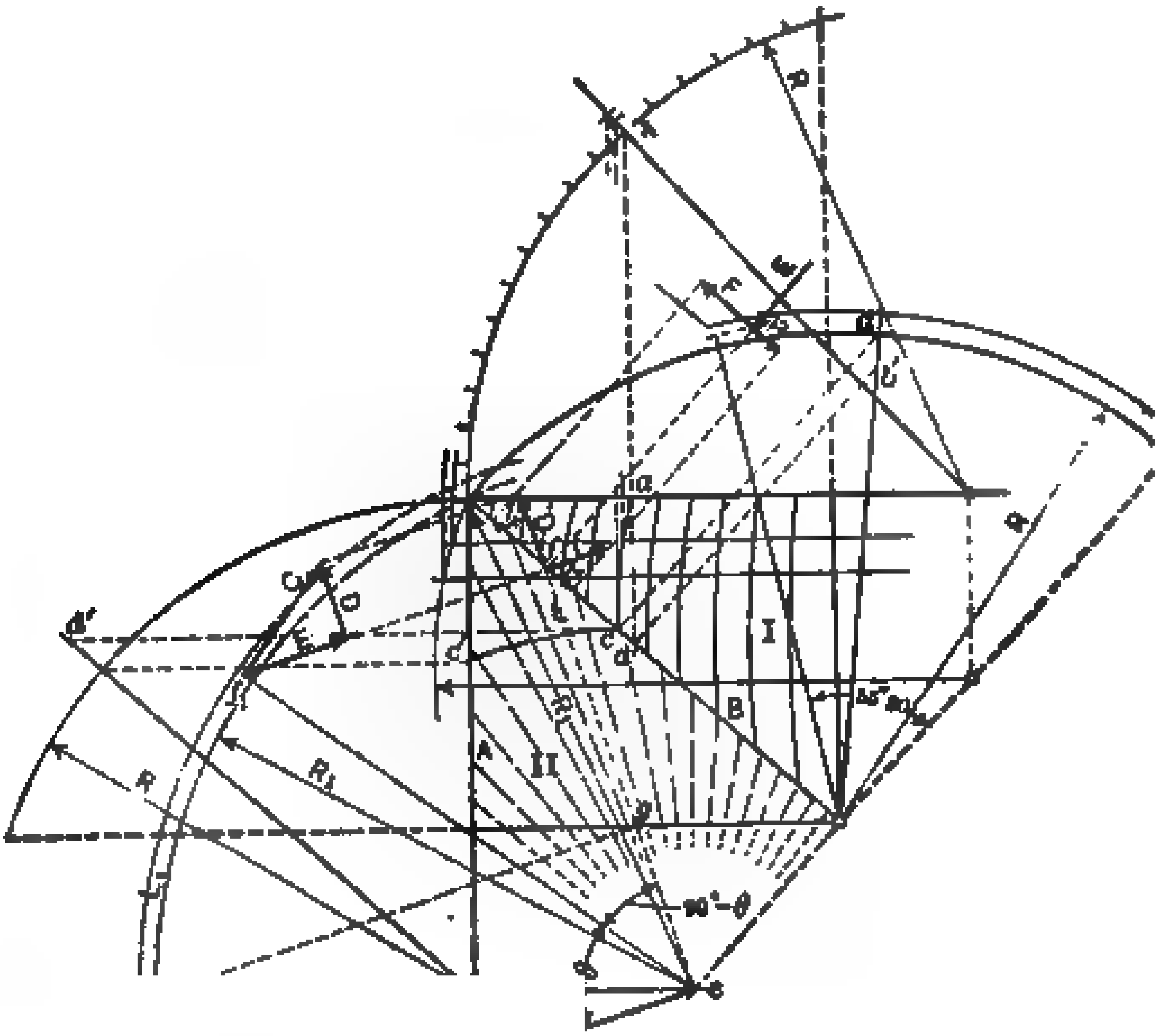


Fig. 13. Determination of Stresses in Vault-ribs

the spring of the diagonal rib B The thrust of each LUNE on the ribs is t
found as shown for lune L

Example. Let the radius R_1, Fig 13 II, be 25 feet, and let the shell be
thick, constructed of stone, and weighing 125 lb per cu ft The angle θ (by me
urement) is 54° 30′, and the angle α is 18° 30′. These are found by project
up from the point of intersection f of the center line of the lune L and the cen
of the rib B, and the intersection g of the center line of the lune L and the cro
line of the vault, to f_1 and g_1, respectively, on the vertical projection L_1 of
lune L.

Using the same notation, equations, and curves as were derived for smoo
SHELL DOMES (page 1214), it is found from Plate II, with $\frac{\sigma}{a} = 0$ and $\alpha = 18°$,
that

$$W_{l-\phi} = -125\,(4/12)\,(25)^2(0.33) = -8\,594 \text{ lb}$$

and that

$$n = \frac{-8\,594}{2\pi(125)(4/12)(25)^2} = -0.0525$$

From Plate I it is found that the CRITICAL ANGLE, for values of $\frac{\sigma}{a} = 0$

.0525, is 55° 30′, and the vaulting should be back-filled as high as this, n.

Plate III, with $\frac{cr}{a} = 0$. and $n = -0.0525$, it is found that at $\theta = 54° 30'$

$$U = (125 \times 4/12 \times 25)(-0.079 + 0.63) = 573 \text{ lb}$$

urement, the width of LUNE L at f is 2 ft; hence the total TANGENTIAL C (Fig. 13), is $2 \times 573 = 1\,146$ lb. The HORIZONTAL COMPONENT D 6 655 lb, and the COMPONENT F (along the rib B) of D is 5 750 lb. The COMPONENT E of C is 9 339 lb.

LUE OF T is found from Plate IV to be $125 \times 4/12 \times (25)^2 \times (-0.004$ $= 7\,570$ lb, and the COMPONENT H (along the rib B) of T is 3 630 lb. RUSTS acting on the rib B of the other LUNES above L are found in the ', and the portion of rib above the back fill investigated as an arch. that portion of the rib below the web is not indicated.

lts with semicircular diagonals of about 33-ft span, the ribs should be 10 in wide and from 10 to 14 in in total height, and the minimum s of the projecting portions of the ribs below the webs, for smaller ould be 3½ in width and 6 in in height.*

ults. TILE VAULTS, as built by the R. Guastavino Company, are d of tiles, from 6 by 12 to 24 in in plan, and 1 in in thickness, and veral layers so as to make a solid, thin shell that is both light and Because of the overlapping of the tiles, the shell has considerable ESISTANCE, and the vaults are practically MONOLITHIC. It is due to the lightness of the construction that the thrusts and the weight of tructure are materially reduced. Ordinarily a finished ACOUSTIC TILE, rough CONSTRUCTIONAL TILE, is used for the exposed surfaces.

Vaults. Vaulting in buildings of moderate cost is frequently l by suspending from the roof-trusses STEEL OR WOODEN FRAMES th and plaster. The roof-trusses must in this case be designed to lirect loads of the FRAMED VAULTING, which must be of the required d shape to carry and fit the plastered surfaces.

* Handbuch der Architektur.

PART III

USEFUL INFORMATION

FOR

ITECTS, DRAUGHTSMEN, BUILDERS, AND SUPERINTENDENTS

AND ALL WHO HAVE TO DO WITH THE BUILDING TRADES

The editor has arranged the information in Part III in the following

g and Ventilation.
ys.
lics, Plumbing and Drainage, Gas and Gas-Piping.
g and Illumination of Buildings.
Work for Buildings.
tural Acoustics.
, Quantities, and Miscellaneous Data on Building Materials.
ons and Data Useful in the Preparation of Drawings and Specifica-

neous Information for Architects and Builders.
of Architectural and Technical Terms.
efinitions of Architectural Terms.

HEATING AND VENTILATION OF BUILDINGS *

By

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Physical Units and the Measurement of Heat

m of Units. In this country the system of units in general use by is known as the FOOT-POUND-SECOND SYSTEM, and the following defini- examples will show the significance of each.

tion of Units Employed. The UNIT OF TIME is the second, which is $\frac{1}{86\,400}$ part of the mean solar day. t = time. Time is also expressed in und hours.

ngth. The UNIT OF LENGTH is the foot = 0.3048 meter.

eight. The UNIT OF WEIGHT is the pound = 0.4532 kilogram.

rea. The UNIT OF AREA is the square foot. The unit often used is the square inch.

olume. The UNIT OF VOLUME is the cubic foot. Volume = area $\times$ length $= A \times L$.

tions involving the quantity of air required Q is often used for cubic

e. The volume displaced per stroke by the plunger of a pump, if the is 6 in and the stroke is 12 in, is $\frac{1}{4} \times \pi \times 6^2 \times 12 = 339.29$ cu in, ı ft.

lunger makes 30 WORKING strokes (not revolutions) per minute, then r-DISPLACEMENT per minute is $0.196 \times 30 = 5.88$ cu ft. One United lon = 231 cu in = 0.1336 cu ft. This pump will therefore theoret- ver 5.88/0.1336, or 44 gal per minute. The actual delivery of the be 10 to 15% less, owing to the SLIP, which is the leakage back through valves, around the plunger, and that due to imperfect filling of the nder on the suction-stroke.

. D = density. The weight of a unit volume (1 cu ft) of a sub- lled its DENSITY. The density of water at 70° F. is 62.3 lb per cubic density of air at 70° is .075 lb per cubic foot. The pump in the pre- mple would, therefore, handle 5.88×62.3 or 366 lb of water per

ter-end of the pump is operated by a steam-cylinder having a dis- of 0.349 cu ft per stroke, and takes steam at the same pressure for ke as in the DIRECT-ACTING type and if we assume that the steam- 100-lb gauge, we find from the steam-table (Table I), that the team at this pressure is 0.2565 lb. The STEAM-CONSUMPTION of the efore, would be $0.2565 \times 0.349 \times 30 \times 60 = 161.6$ lb per hour, . A fan handling 10 000 cu ft per minute of air at 70° F, delivers 75 = 100 lb per minute.

v = velocity. The RATE OF MOTION of a body is measured by the sed over in a unit time. Velocity is expressed in feet per second.

the data of this section has been condensed from Vol. I of Mechanical Buildings, by Harding and Willard, published by John Wiley & Sons, Inc.

Energy or Work. U = ENERGY OR WORK. The UNIT OF WORK is the f pound, and is the quantity of energy expended or the work performed by a f of 1 lb moving through a distance of 1 ft in the line of action of the force.

Power is the RATE OF DOING WORK. Note that POWER involves the fa TIME and is equal to the amount of work done divided by the time require do this work.

Horse-Power. h.p. = HORSE-POWER. The UNIT OF POWER is the ho power and is the performance of work at the rate of 550 ft-lb per second 33 000 ft-lb per minute.

Example. Required the theoretical work and horse-power developed by water-end of the pump in the preceding example, assuming that the head or height pumped against is 200 ft, and that no frictional resistance is to be ov come.

The work Um performed per minute is the lifting of the weight of water, 366 lb per min, through a height of 200 ft and is $Um = 366 \times 200 = 73\ 200$ f per min. The h.p. $= Um/33\ 000 = 73\ 200/33\ 000 = 2.22$.

The actual power required will be somewhat greater, as the force requi to overcome frictional resistance, etc., has been neglected.

Equivalent Values of Electrical and Mechanical Units

1 horse-power..	746 watts 0.746 kilowatt 33 000 ft-lb per min 550 ft-lb per sec 2 546 Btu per hr	1 kilowatt	1 000 watts 1.34 h.p. 2 654 200 ft-lb per h 44 240 ft-lb per min 737.6 ft-lb per sec 3 414.5 Btu per hr

Measurement of Pressure. It is customary to measure PRESSURE by me of GAUGES which, in reality, only indicate the difference between the pres being measured and the pressure of the atmosphere, BAROMETRIC PRESSU at the same time and place. These gauges may indicate either a higher or lo pressure than that of the atmosphere; in the former case they are know PRESSURE-GAUGES and in the latter as VACUUM-GAUGES or DRAFT-GAUGES.

Pressure-Gauges and Vacuum-Gauges. The most common type of pr sure-gauge (Fig. 1) is provided with a flexible hollow brass tube of oval cr section known as BOURDON TUBE. When s jected to pressure, this tube tends to straigh out; and this causes a sector of a gear to m with a small pinion, which is on the same sh with the indicating hand or pointer, and ro the latter a corresponding amount. The poi is placed just in front of a graduated dial, shown in the figure, from which the press may be read in suitable pressure-units, such pounds per square inch. These gauges may be used for indicating vacuum, or a pressure than that of the atmosphere.

Fig. 1. Single-spring Pressure-gauge. Interior View

Draft-Gauges. The measurement of press but slightly above or below the atmosph pressure, barometric pressure, is usually accomplished by the use of a DRA GAUGE. This is essentially a U tube, containing either water, kerosene, a hol, or mercury, mounted upon a graduated scale, and reading either in inc of fluid or in pounds or ounces per square inch. Since the pressure indica

ferential one, due to the left-hand leg being open to the air, the reading : obtained by adding the depression in the left-hand leg to the elevation ight-hand leg, using zero as the reference-point in both cases.

neters. The PRESSURE OF THE ATMOSPHERE is usually measured by a IAL BAROMETER (Fig. 2), which, in its simplest form, consists of a glass out 3 ft long, closed at one end. After led with mercury it is inverted in a shallow mercury. The pressure of the atmosphere evel maintains the mercury-column in the out 30 in above the level in the bath or The barometric height or length of this of mercury varies with the altitude above sea-level. When the mercury in the tube t in the cistern rises in corresponding pro- and vice versa, so that there is an ever- relation between the level of the mercury ube and the mercury in the cistern, which he accuracy of the readings. It is, there- essary, before reading the height of the column on the stem of the barometer by f a movable vernier, to adjust the level of ury in the cistern. All standard or observ- ometers of the mercurial type have this nt. Barometers of other types, such as ROID BAROMETER, must be frequently l with a standard mercurial barometer in check the accuracy of their readings.

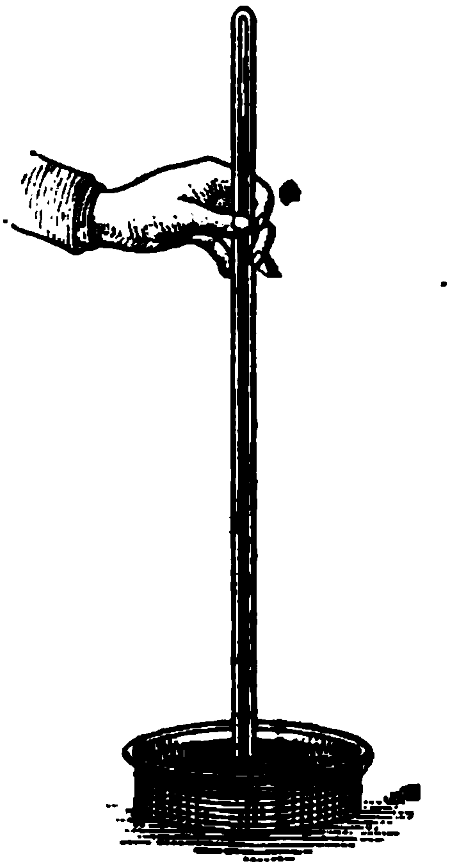


Fig. 2. Simple Barometer

etric Pressure. By BAROMETRIC HEIGHT is meant the height of a f pure mercury at 32° F. which just balances the pressure of the atmos- the time and place of the observation. The STANDARD OR NORMAL IIC PRESSURE is defined as the pressure of a column of pure mercury (29.92 in) high at 32° F. This is the normal barometric pressure at 5° and at sea-level. Since the weight of 1 cu in of mercury under e conditions is 0.491 lb, the normal barometric pressure = height of olumn × weight per cubic inch = 29.92 × 0.491, or 14.7 lb per iis pressure of 14.7 lb per sq in is known as the ABSOLUTE PRESSURE nosphere at latitude 45° and at sea-level. Now, since the ordinary auge measures only pressures above or below that of the atmosphere, sary to ADD THE BAROMETRIC PRESSURE at the place in question TO E-READING TO OBTAIN THE TOTAL ABSOLUTE PRESSURE corresponding sure indicated by the gauge. That is, absolute pressure = barometric - gauge-pressure.

Heat

on of Heat. HEAT IS A FORM OF ENERGY. It is, in fact, the kinetic ial energy of the molecules of which all substances, whether solid, aseous, are composed. Whenever the vibratory motion of the mole- osing a body of given mass is increased from any cause the THERMAL IERGY is increased. The temperature of the body rises, its SENSIBLE ases, and the body feels warmer.

ement of Temperature. Thermometry. INTENSITY OF HEAT is by THERMOMETERS and PYROMETERS, the latter being used for high es above from 400° to 500° F. In engineering work mercurial

thermometers are very largely employed. These depend upon the unif expansion of mercury to indicate changes in temperature. The UNIT OF MEASU MENT is called a DEGREE, and is capable of very exact determination, provi that two points, at which the heat-intensity is always constant, can be use bases or references for calibration. The melting-point of ice and boiling-p of water at atmospheric pressure are usually selected as bases, and the unif expansion of the mercury between these two points is indicated on a s divided into 180, 100, or 80 divisions. (Fig. 3.) Each of these division known as a DEGREE and the scales used are known spectively as FAHRENHEIT, CENTIGRADE or CELS and REAUMUR. The Fahrenheit is used almost ex sively in engineering in this country.

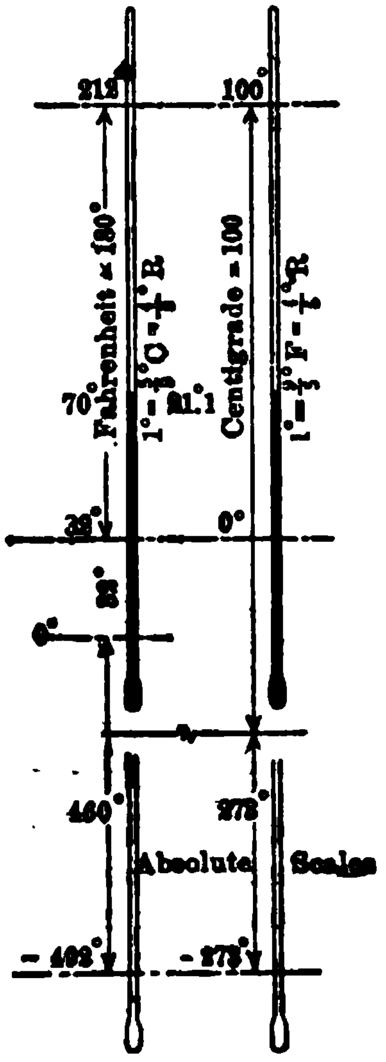


Fig. 3. Fahrenheit and Centigrade Thermometers

Absolute Temperature. In addition to the t temperature-scales already described, physicists em what is known as the ABSOLUTE SCALE OF TEMPE TURES, based on the so-called ABSOLUTE ZERO OF T PERATURE, at which point no molecular vibration ex This zero is conceived as 491.6° F. below the melti point of ice (32° F.), it having been discovered tha ideal perfect gas would change in volume by 1/4 of its volume at 32° F. for each 1° change in its t perature, at constant pressure. Thus, if 491.6 c of gas, measured at 32° F., is cooled 20° F. at const pressure, the new volume will be 471.6 cu ft. I only necessary to add 491.6 − 32, or 459.6, to actual thermometer-reading to get the absolute t perature. That is, $T = t + 459.6$, where T = abso temperature, and t = actual thermometer-reading the Fahrenheit-scale. For engineering-work, 460° used rather than 459.6°. For the Centigrade scale relation is $T = t + 273.1$.

Measurement of Heat-Quantity. Calorime HEAT MAY BE MEASURED, since it is a form of ener in any of the usual energy-units, as the JOULE, FO POUND, or HORSE-POWER HOUR. It is the custom, h ever, to use for this purpose a special unit more rea applicable to heat-changes. This unit in the English system is known as BRITISH THERMAL UNIT (Btu), and is the amount of heat required to raise 1 l water from 63° to 64° F. For all practical purposes in ordinary calculatio Btu is the amount of heat required to raise 1 lb of water 1° F.

Specific Heat. It is a well-known fact that equal quantities of heat raise equal weights of different substances a different number of degrees, depe ing on the nature of the substances. This property of matter is know SPECIFIC HEAT, and for any substance can be expressed as the number of required to raise or lower the temperature of 1 lb 1° F., at some given t perature. It is also customary to make use of the mean or average value a certain temperature-interval. Two specific heats are recognized, one kn as the TRUE SPECIFIC HEAT, measured at the temperature stated, and the o as the MEAN SPECIFIC HEAT, which is the average value between the temperat under consideration. The specific heat of air at constant pressure is 0.24.

Relation between Units of Energy and Power. Since the various form energy, heat, mechanical energy, electrical energy, etc., are mutually conv ible, there must be definite numerical relations between the various units

ess energy. As determined by various physicists the relation between
ı and the ft-lb is

$$1 \text{ Btu} = 777.64 \text{ ft-lb}$$

umber 777.64 is called the MECHANICAL EQUIVALENT OF HEAT and is
l by J. For ordinary use the value 778 may be taken. Another con-
relation is,

$$1 \text{ h.p.} = 2\,564 \text{ Btu per hr}$$

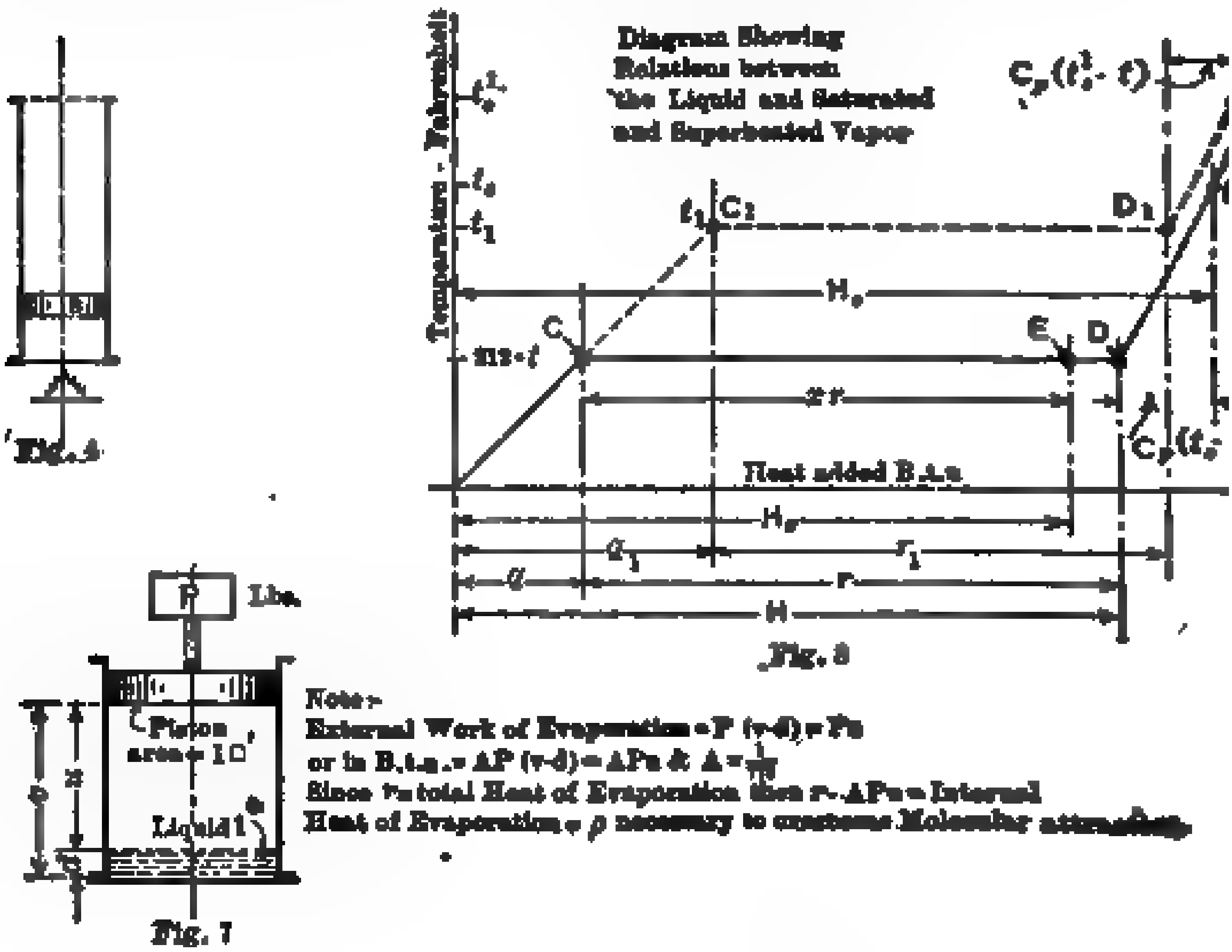
Steam

rties of Steam. Steam is water-vapor, which exists in the vaporous
n because sufficient heat has been added to the water, from which the
as been formed, to supply the latent heat of evaporation, and to change
id into a vapor. This change of state takes place at a definite and
temperature, which is determined solely by the pressure of the steam.
e in pressure will always be accompanied by a change in the tempera-
which ebullition or boiling will occur, and there will be a corresponding
n the latent heat. The properties of steam, together with other char-
s, are tabulated in the steam-tables. (See Table I.) Steam in con-
t the water from which it has been generated is known as SATURATED
nd may be known as DRY SATURATED STEAM, or as WET SATURATED
The latter contains more or less actual water in the form of mist or
as it is called. If dry, saturated steam be heated, and the pressure
ed the same as when it was vaporized, its temperature will increase
ll become SUPERHEATED; that is, its temperature will be higher than
turated steam at the same pressure.

le and Latent Heat. Whenever heat is added to a substance, without
state, its temperature is raised, and the heat thus added is known as
HEAT, as, for example, the heat added to water the temperature of
etween 50° and 140° F. Sensible-heat changes, as already stated, are
by the thermometer. Heat may be added to a body without any
temperature provided a change of state from solid to liquid or from
apor takes place, and the heat thus added is known as LATENT HEAT.
change is from solid to liquid, as from ice to water, this heat is known
ENT HEAT OF FUSION. At atmospheric pressure ice melts at 32° F.,
ent heat is 144 Btu per lb. When the change is from liquid to vapor,
ter to steam, the heat required to effect the change is known as the
AT OF EVAPORATION. At atmospheric pressure water evaporates at
nd the latent heat is 971.7 Btu per lb. A conception of the relation
e properties or characteristics of steam, and the manner in which the
state, temperature and pressure are brought about, is described in the
aragraphs.

ion of Steam. Consider a frictionless cylinder (Fig. 4), containing
er at 32° F. Also consider the pressure of the atmosphere to be 14.7
and to be replaced by that of the piston B. When heat is applied
der the temperature of the water rises until the boiling-point, 212° F.,
The heat necessary to raise the temperature from 32° F. to the
it is known as the HEAT OF THE LIQUID or SENSIBLE HEAT, and is de-
e symbol Q. This condition is denoted in Fig. 8 by the point C.
e specific heat of water between 32° F. and 212° F. is 1; hence the
British thermal units (Btu) necessary to raise the temperature of
his amount is 212 − 32 or 180 Btu.
re heat is added the water begins to evaporate and expand at con-

stant temperature until, as in Fig. 6, the water is entirely changed into steam. This condition is also shown in Fig. 8, by the point D. The heat thus added known as the LATENT HEAT OF EVAPORATION and is denoted by the symbol This heat r is subdivided into two parts. (See Fig. 7.) First the attractic between the molecules must be broken down. This is known as the INTERNA LATENT HEAT and is denoted by the symbol ρ. Next the external resistan must be overcome, the weight P being raised against gravity. The heat th added is known as EXTERNAL LATENT HEAT and is designated by the symbo APu, where u is the change in volume, in cu ft of 1 lb of water, A is 1/778, an



Figs. 4 to 8. Diagrams Explaining the Generation of Steam

P is the pressure of the atmosphere in pounds per square feet (barometr pressure). It is evident then that the latent heat

$$r = \rho + APu, \text{ or } \rho = r - APu$$

The term APu is the heat-equivalent of the work performed for the change i volume from water to steam.

The heat added from the starting point (32° F), is known as TOTAL HE (H), or $q + r = H$. If more heat is added, the pressure remaining constan the temperature of the steam rises and the steam becomes what is known SUPERHEATED STEAM. The heat added is equal to the MEAN SPECIFIC HE (C_p) of the steam, times the change in temperature ($t_s - 212$). The speci heat of steam is the Btu, or heat, required to raise the temperature of 1 lb of th steam 1° F. Since the specific heat of steam is less than that of water, th slope of this line becomes greater than that of the water-line The point now located at t_s (Fig 6), and the steam has increased in volume in the cylind (Fig 5), until the piston occupies the dotted position B'.

If instead of the above condition of pressure, additional pressure is adde

Table I. Properties of Saturated Steam. *

G. A. Goodenough

e pressure	Temperature, deg. F.	Volume, cu ft per lb	Weight, lb per cu ft	Heat-content in Btu		Latent heat in Btu	
Lb per sq in				of liquid	of vapor	of vaporization	Internal
p	t	v	d	q	H	r	ρ
2	126.10	173.6	0.00576	94.02	1 116.2	1 022.2	957.9
4	152.99	90.6	0.01104	120.9	1 127.9	1 007.0	939.9
6	170.07	62.0	0.01614	137.9	1 135.0	997.1	928.2
8	182.87	47.35	0.02112	150.8	1 140.3	989.5	919.4
10	193.21	38.43	0.02602	161.1	1 144.4	983.3	912.2
12	201.96	32.41	0.03086	169.9	1 147.9	978.0	906.0
14.74	212.13	26.75	0.03739	180.1	1 151.8	971.7	898.8
16	216.3	24.76	0.04038	184.3	1 153.4	969.1	895.8
18	222.4	22.18	0.04508	190.5	1 155.7	965.2	891.4
20	228.0	20.10	0.04976	196.0	1 157.7	961.7	887.3
22	233.1	18.38	0.0544	201.2	1 159.6	958.4	883.6
24	237.8	16.95	0.0590	206.0	1 161.3	955.3	880.1
26	242.2	15.73	0.0636	210.4	1 162.8	952.4	876.8
28	246.4	14.67	0.0681	214.6	1 164.3	949.7	873.7
30	250.3	13.76	0.0727	218.6	1 165.7	947.1	870.7
32	254.0	12.95	0.0772	222.4	1 166.9	944.6	867.9
4	257.6	12.24	0.0818	225.9	1 168.1	942.2	865.2
6	260.9	11.60	0.0862	229.4	1 169.2	939.9	862.7
8	264.2	11.03	0.0907	232.6	1 170.3	937.7	860.2
[illegible]	267.2	10.51	0.0951	235.8	1 171.3	935.5	857.8
[illegible]	281.0	8.53	0.1173	249.8	1 175.6	925.9	847.1
[illegible]	285.9	7.93	0.1261	254.7	1 177.1	922.4	843.2
	292.7	7.18	0.1392	261.7	1 179.1	917.4	837.8
	296.9	6.76	0.1479	266.1	1 180.3	914.3	834.3
	302.9	6.22	0.1609	272.2	1 182.0	909.8	829.5
	306.7	5.90	0.1695	276.1	1 183.0	906.9	826.4
	312.0	5.48	0.1824	281.6	1 184.4	902.8	821.9
	315.4	5.23	0.1910	285.1	1 185.3	900.2	819.1
	320.3	4.905	0.2039	290.1	1 186.5	986.4	815.0
	323.3	4.709	0.2124	293.3	1 187.3	894.0	812.4
	327.8	4.442	0.2251	297.9	1 188.4	890.5	808.6
	330.7	4.279	0.2337	300.9	1 189.0	888.2	806.1
	334.8	4.057	0.2465	305.1	1 190.0	884.8	802.6
	337.4	3.921	0.2550	307.9	1 190.6	882.7	800.3
	341.3	3.735	0.2678	311.9	1 191.4	879.5	796.9
	343.7	3.620	0.2762	314.4	1 191.9	877.5	794.8
	347.4	3.461	0.2889	318.2	1 192.6	874.4	791.6
	349.7	3.363	0.2973	320.6	1 193.1	872.5	789.5
	353.1	3.226	0.3100	324.2	1 193.7	869.6	786.4
	355.3	3.140	0.3184	326.5	1 194.1	867.7	784.5
	358.5	3.020	0.3311	329.8	1 194.7	864.9	781.6
	360.5	2.945	0.3396	332.0	1 195.1	863.1	779.7
	363.6	2.839	0.3522	335.2	1 195.7	860.5	776.9
	365.6	2.773	0.3606	337.3	1 196.0	858.7	775.1
	368.5	2.679	0.3733	340.3	1 196.5	856.2	772.4
	370.4	2.620	0.3817	342.3	1 196.8	854.5	770.6
	373.1	2.536	0.3943	345.2	1 197.2	852.0	768.0
	377.6	2.408	0.4154	350.0	1 197.9	847.9	763.9
	381.9	2.292	0.4364	354.5	1 198.5	844.0	759.8

rom original tables published by John Wiley & Sons, Inc.

as shown by the weight W in Fig. 6, the temperature of the boiling-point will raised from the temperature of 212° F. to some other point, as t_1 (Fig. 8). may be seen by this figure, the sensible heat q has been increased to q_1. Wh more heat is added the water is evaporated at the temperature t_1, and if he again be added the SATURATED STEAM will become SUPERHEATED STEAM.

Quality of Steam. The proportion of the DRY STEAM, per pound of ste delivered by the boiler, is known as the QUALITY OF THE STEAM and is rep sented by the symbol x, and the heat (H_x) contained in the steam above 32° is $q + xr$; the state-point is located at E (Fig. 8).

Specific Volume and Density. The volume of a pound of steam is kno as the SPECIFIC VOLUME v, and as may be seen by comparing Figs. 5 and 6, creases as the pressure increases. The reciprocal of this, or the weight of ste per cubic foot, is known as the DENSITY, and is denoted by d or $1/v$.

Entropy. Another quantity known as ENTROPY is made use of in calcu tions relating to steam-engines and turbines, and is defined as the ratio obtain by dividing the quantity of heat added to a substance by the absolute tempe ture at which it is added.

The Total Heat, H, of a dry, saturated vapor for any pressure and te perature is the sum of the heats required to raise the temperature of one pou of the liquid from the freezing-point to the given temperature and correspondi pressure and ENTIRELY VAPORIZE IT at this pressure. For this case $x =$ and consequently

$$H = (\rho + APu) + q = r + q$$

The total heat (H_x) of wet vapor at any pressure and temperature is

$$H_x = xr + q$$

It is manifestly incorrect to say that this is the heat in the vapor, as the A is not the heat in the vapor, but the external work performed by the vapor wh evaporating.

Superheated Steam or Vapor. Superheated steam is defined as water-va which has been heated out of contact with its liquid, until its temperatur higher than that of saturated vapor at the same pressure.

The heat-content of superheated steam or vapor may be expressed by equation

$$H_s = q + r + C_p(t_s - t) = H + C_p(t_s - t)$$

where t_s is the temperature of superheated vapor, t the temperature of satura vapor at the corresponding pressure, q the heat of the liquid at t, an the heat of vaporization at temperature t. C_p is the mean specific heat superheated vapor (approximately 0.50), H the total heat of 1 lb of dry urated steam, and H_s the total heat of 1 lb of superheated steam.

Properties of Air

Charles' Law. Charles' Law refers to the relation between pressure, vol and temperature of a gas, and may be stated as follows. The volume given weight of gas varies directly as the absolute temperature at cons pressure, and the pressure varies directly as the absolute temperature at cons volume. Hence, when heat is added at constant volume V_c, this equation res

$$\frac{P_2}{P_1} = \frac{T_2}{T_1}$$

: temperature-range, at constant pressure P_0, the relation is

$$\frac{V_2}{V_1} = \frac{T_2}{T_1}$$

or any weight of gas M, since volume is proportional to weight at ıme and temperature,

$$PV = MRT$$

ıaracteristic equation for a perfect gas. In this formula

bsolute pressure of the gas in pounds per square foot = 2116.8 ıtmospheric pressure);
'olume of the weight M in cubic feet;
veight in pounds of the gas taken;
ıstant depending on the nature of the gas = 53.37 for air;
ıbsolute temperature in degrees Fahrenheit (t + 459.6).

Table II. Properties of Dry Air

Barometric pressure, 29.921 in. Specific heat, 0.24

...re ... ...t	Weight per cubic foot in pounds	Per cent of volume at 70° Fahrenheit	Btu absorbed by one cubic foot dry air per degree Fahrenheit	Cubic feet of dry air warmed one degree per Btu
	0.08636	0.8680	0.02080	48.08
	0.08453	0.8867	0.02039	49.08
	0.08276	0.9057	0.01998	50.05
	0.08107	0.9246	0.01957	51.10
	0.07945	0.9434	0.01919	52.11
	0.07788	0.9624	0.01881	53.17
	0.07640	0.9811	0.01846	54.18
	0.07495	1.0000	0.01812	55.19
	0.07356	1.0190	0.01779	56.21
	0.07222	1.0380	0.01747	57.25
	0.07093	1.0570	0.01716	58.28
	0.06968	1.0756	0.01687	59.28
	0.06848	1.0945	0.01659	60.28
	0.06732	1.1133	0.01631	61.32
	0.06620	1.1320	0.01605	62.31
	0.06510	1.1512	0.01578	63.37
	0.06406	1.1700	0.01554	64.35
	0.06304	1.1890	0.01530	65.36
	0.06205	1.2080	0.01506	66.40
	0.06110	1.2270	0.01484	67.40
	0.06018	1.2455	0.01462	68.41
	0.05673	1.3212	0.01380	72.46
	0.05225	1.4345	0.01274	78.50
	0.04903	1.5288	0.01197	83.55
	0.04618	1.6230	0.01130	88.50
	0.04364	1.7177	0.01070	93.46
	0.04138	1.8113	0.01018	98.24
	0.03934	1.9060	0.00967	103.42
	0.03746	2.0010	0.00923	108.35
	0.03423	2.1900	0.00847	118.07

A PERFECT GAS conforms exactly to the above equation, and while no gases PERFECT in this sense, they conform so nearly that the above equation app to most engineering-computations. The volume of 1 lb of air, known as SPECIFIC VOLUME, at any temperature and pressure, can be found at once the equation

$$V = (53.37 \times T)/P$$

Estimating Heating Requirements of Buildings

Heat Required and Supplied. The amount of heat, measured in Btu to supplied by the heating-apparatus to a building to maintain the inside tempe ture above that of the outside, commonly termed HEAT-LOSSES, is:

(a) The heat required to offset the heat-transmission of the walls, ceilin roof, and floor. This loss of heat depends upon the type and materials of c struction used and the temperature-difference to be maintained between inside and the outside of the building.

(b) The heat required to warm the air entering the building from the outsi either by infiltration or purposely introduced for ventilation.

(c) The heat supplied by persons, lights, machinery and motors, which m be deducted from the sum of items (a) and (b) to obtain the net amount of h to be supplied by the heating-apparatus. (Item (c) is usually not considere

It is customary in all calculations connected with the design of heati installations to base the estimate on the amount of heat per hour to be suppl by the apparatus. The total heat to be supplied per hour is H = [(item a) (item b) − (item c)] Btu. The method in use for the calculation of the vari items above mentioned will now be taken up and discussed in the order give

Temperatures. The inside temperature to be maintained and the air quired for ventilation for various classes of work are discussed under Ventilati to which the reader is referred. The outside temperature for which the heati installation should be designed is fixed by the lowest outside temperature t is liable to continue for several days during the heating-season.

Usual Inside Temperature Specified

Kind of buildings	Degrees F.
Public buildings	68–72
Factories	65
Machine-shops	60–65
Foundries, boiler-shops, etc.	50–60
Residences	70
Bath-rooms	85
Schools	70
Hospitals	72–75
Paint-shops	80

In designing the heating-system a temperature of from 10° to 15° F. hi than the lowest recorded temperature is recommended to be used for the side temperature.

Heat-Transmission of Walls, Ceilings, Roofs, Floors, etc. (a) The h loss through building-construction is dependent upon the character of material, thickness and character of the surfaces, and the velocity of th over the surfaces. Numerous tests have been conducted by various ex menters to determine accurately the heat-transmission of various type

Table III. Outside Temperatures

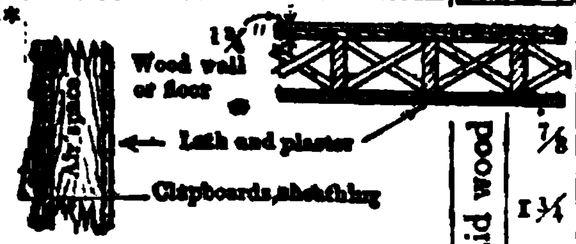
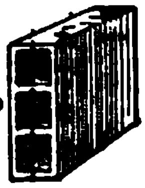
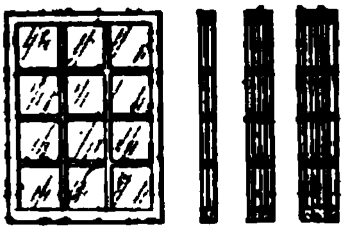
nd Average Temperatures in the United States. All stated in Fahrenhei degrees and compiled from United States Weather Bureau Records

City	Lowest	Aver-age.*	State	City	Lowest	Aver-age *
Mobile.......	− 1	57.7	Neb. .	North Platte..	−35	34.6
Montgomery..	5	56.1		Lincoln........	−29	35.8
Flagstaff......	−21	34.8	Nev. .	Carson City...	−22	......
Phoenix......	22	58.9		Winnemucca..	−28	37.9
Fort Smith....	−15	49.5	N. H	Concord......	−35	33.1
Little Rock...	−12	52.0	N. J..	Atlantic City. .	− 7	41.6
San Diego....	32	57.2	N. Y.	Saranac Lake..	−38	34.1
Independence..	10	48.7		New York City	− 6	40.1
Denver.......	−29	38.4	N. M.	Roswell.......	−14	48.9
Grand Junction	−16	39.2		Santa Fé......	−13	38.0
Southington...	−19	36.3	N. C.	Hatteras......	8	53.3
Washington...	−15	42.9		Charlotte.....	− 5	49.8
Jupiter.......	24	69.8	N. D.	Devil's Lake. .	−51	18.9
Jacksonville...	10	60.9		Bismarck.....	−44	23.5
Savannah.....	8	57.2	Ohio .	Toledo.......	−16	36.8
Atlanta.......	− 8	51.4		Columbus.....	−20	39.8
Boise.........	−28	39.6	Okla .	Oklahoma.....	−17	47.1
Lewiston......	−18	42.5	Ore. .	Baker City....	−20	34.1
Chicago......	−23	35.9		Portland......	− 2	45.4
Springfield....	−22	39.0	Pa...	Pittsburgh....	−20	40.8
ndianapolis...	−25	40.4		Philadelphia...	− 6	41.8
Evansville....	−15	44.1	R. I. .	Providence....	− 9	37.5
ioux City....	−31	32.1		Rock Island...	− 4	39.7
Keokuk......	−26	37.6	S. C. .	Charleston....	7	56.9
Dodge City...	−26	....		Columbia.....	2	53.5
Wichita......	−22	42.9	S. D..	Huron........	−43	25.9
ouisville.....	−20	45.0		Yankton......	−32	31.2
ew Orleans..	7	60.5	Tenn.	Knoxville.....	−16	47.0
hreveport....	− 5	55.7		Memphis.....	− 9	50.7
astport......	−21	31.1	Tex. .	Corpus Christi.	11	62.7
ortland......	−17	33.5		Fort Worth...	− 8	49.5
altimore.....	− 7	43.3	Utah.	Salt Lake City.	20	39.7
oston.......	−13	37.2	Vt...	Northfield....	−32	27.8
lpena.......	−27	29.1	Va...	Cape Henry...	5	48.6
etroit.......	−24	35.3		Lynchburg....	− 5	45.2
uluth.......	−41	25.5	Wash.	Seattle.......	3	44.3
inneapolis...	−33	28.4		Spokane......	−30	37.0
eridian.....	− 6	53.9	W.Va.	Parkersburg...	−27	41.9
icksburg.....	− 1	56.0		Elkins........	−21	38.8
ringfield....	−29	43.0	Wis. .	La Crosse.....	−43	31.2
annibal.....	−20	39.7		Milwaukee....	−25	32.4
avre........	−55	27.7	Wyo .	Cheyenne.....	−38	33.7
elena.......	−42	30.9		Lander.......	−36	29.0

* Average is taken from October 1 to May 1.

n. The following table represents the results of the experiment by Harding and Willard in this connection, based on an averag d-movement of approximately 15 miles per hour:

Table IV. Heat-Transmission of Building-Construction

Construction	Thickness, in	Btu transmitted per square foot per hour — Temperature-difference 1°	20°	40°	60°	70°	80°
 Plain brick wall	9	.363	7.3	14.5	21.8	25.4	29.
	13	.281	5.6	11.2	16.9	19.7	22.
	18	.220	4.4	8.8	13.2	15.4	17.
	24	.174	3.5	7.0	10.4	12.2	13.
 Brick wall and air-space, furred and plastered	9	.217	4.3	8.7	13.0	15.2	17.
	13	.185	3.7	7.4	11.1	13.0	14.
	18	.156	3.1	6.2	9.4	10.9	12.
	24	.132	2.6	5.3	7.9	9.2	10.
*  Wood wall or floor; Lath and plaster; Clapboards, sheathing	1⅜″	.20	4.0	8.0	12.0	14.0	16.
	Solid wood ⅞	.547	10.9	21.9	32.8	38.3	43.
	Solid wood 1¾	.370	7.4	14.8	22.2	25.9	29.
	Solid wood 2⅝	.279	5.6	11.2	16.7	19.5	22.
 Hollow tile, ¾-in plaster on both sides	2	.409	8.2	16.4	24.5	28.6	32.
	4	.325	6.5	13.0	19.5	22.8	26.
	6	.281	5.6	11.2	16.9	19.7	22.
Concrete walls 	2	.784	15.7	31.4	47.0	54.9	62.
	3	.714	14.3	24.6	42.8	50.0	57.
	4	.655	13.1	26.2	39.3	45.9	52.
	6	.563	11.3	22.5	33.8	39.4	45.
	For 3-in concrete covered with slag roofing, deduct approximately 10% from values stated.						
Windows 	Single	1.126	22.5	45.0	67.6	78.8	90.
	Double	.450	9.0	18.0	27.0	31.5	36.
	Triple	.281	5.6	11.2	16.9	19.7	22.
One air-change per hr cu ft		.018	.360	.720	1.08	1.26	1.4
Btu loss per foot of sash perimeter per hour							
Wooden sash		2.05	41.0	82.0	123	144	16
Wooden sash, metal strip		0.43	8.6	17.2	26	30	3
Hollow metal sash		4.5	90	180	270	315	36
Hollow metal sash, stripped		1.6	32	64	96	112	1

* For lath-and-plaster ceiling with no floor above, double the values given for wood floor with plaster ceiling.

data on the heat-transmission of various types of roofs were st-results of C. L. Norton:

Table V. Heat-Transmission through Roofs

Construction	Btu per sq ft per hour per 1° difference in temperature of still air inside and outside
ab roof 4 in thick with 5-ply tar and felts..	0.134
ab roof 3 ½ in thick with 5-ply tar and felts	0.149
lab roof 3 in thick with 5-ply tar and felts.	0.170
in thick with 5-ply tar and felts..........	0.192
: 3 in thick with 5-ply tar and felts........	0.282
:ta tile 3 in thick with 5-ply tar and felts...	0.348
; in thick with 5-ply tar and felts.........	0.488
4 in thick with 5-ply tar and felts........	0.508
; in thick with 5-ply tar and felts.........	0.575
; in thick with 5-ply tar and felts..........	0.633

mission of stone walls is approximately 50% greater than that thickness. The Btu-loss per foot of sash-perimeter is based on minations by Voorhees and Meyer, Trans. Am. Soc. H. and

ssion of Roofs and Floors. The temperature of the air in under side of a ceiling or roof is found to be higher than the ntained at the breathing-line, at which point the temperature ed; and this is due to the natural tendency of the warmer or ise. It is recommended that an increase of approximately the specified inside temperature for the temperature at the or wall-heights not exceeding 15 ft, and 30% for ceiling-heights n estimating the heat-loss of roofs. Thus, if 65° F. is the spe- perature to be maintained in a room the height of which is ature of the air in contact with the under side of the roof may : 65° + 30%, or 85° F. The loss of heat through the ceiling which a large air-space exists, through partitions between a room, or through the first floor to the cellar, may be estimated n that the warmed rooms give off sufficient heat to maintain of these colder spaces according to the following schedule:

cs under metal or slate roofs.................	14° F.
cs under tile, cement, tar, or gravel roofs......	23° F.
l rooms kept closed.........................	35° F.

nission of floors that are laid directly upon the ground may be assumption that the ground in contact with the under side of pproximate temperature of 50° F. Thus the estimated heat- in concrete floor laid directly upon the ground, assuming an e of 65° F., is

;3 (65 − 50) or 8.4 Btu per square foot per hour

Infiltration. (b) The heat required to warm the outside air

which may enter by LEAKAGE through the cracks or clearances around windo and doors is that required to raise the temperature of the weight of incomi air per hour from the outside to the inside temperature.

Let b = Btu required per hour to heat the incoming air;
t = inside room-temperature in degrees Fahrenheit;
t_0 = outside temperature;
C_p = specific heat of air at constant pressure = 0.24;
d = density of the air at temperature t;
= 0.075 for 70° inside temperature;
= 0.076 for 60° inside temperature;
Q = cubic feet of air per hour entering building by infiltration, measu at temperature t;
W = weight of air per hour entering building by infiltration $= d \times Q$;

Then $b = C_p\,(t - t_0)\,Q \times d = 0.24 \times W \times (t - t_0)$;
$= 1.26\,Q$ for 70° inside temperature;
$= 1.08\,Q$ for 60° inside temperature.

There are two assumptions made by engineers in practice for obtaining th value of Q. The common method in vogue is to assume a certain number o air-changes n, per hour in the cubical contents C, of the room in accordanc with the following table:

Table VI. Number of Air-Changes per Hour

Halls	$n = 3$
Rooms on 1st floor	$n = 2$
Rooms on 2nd floor	$n = 1$
Offices and stores, 1st floor	$n = 2$ to 3
Offices and stores, 2nd floor	$n = 1\frac{1}{2}$ to 2
Churches and public assembly-rooms	$n = \frac{1}{4}$ to 2
Large rooms with small exposure	$n = \frac{1}{2}$ to 1
Factory-buildings	$n = \frac{1}{2}$ to 1

Example. Required the heat-loss, by infiltration, from a room containin 20 000 cu ft, the temperature of which is maintained at 7c° F. in zero weathe the estimated number of air-changes n, being two per hour.

Solution. $Q = 2 \times 20\,000 = 40\,000$ cu ft of air entering per hour measure at 70° F.

$b = 0.018 \times 40\,000 \times (70 - 0) = 50\,400$ Btu per hour.

The other method is to use the estimated amount of air-leaking in the build- ing through the cracks around the sash-perimeter and meeting-rail. The fol- lowing data may be used in this connection and is based on a wind-movement o approximately 20 miles per hour (Voorhees and Meyer Tests).

Plain wooden sash	114	cu ft air per hour per foot perimete
Plain wooden sash, weather-stripped	24	cu ft air per hour per foot perimete
Hollow metal sash	216 to 268	cu ft air per hour per foot perimete
Hollow metal sash, weather-stripped	72 to 150	cu ft air per hour per foot perimete
Copper-covered sash	132	cu ft air per hour per foot perimete

For a room with more than one outside wall use only the sum of the per- imeters of the windows, in the side having the greater number.

office 14 by 16 by 10-ft-high ceiling, has two 3 by 7-ft wooden-
The maintained inside temperature is 70°, and the outside
F. Required the heat-loss by infiltration.

the first method, assuming two air-changes per hour, the loss

$$\cdot 1.26 \times 2 \times (14 \times 16 \times 10) = 5\,645 \text{ Btu per hr}$$

method this loss is:

$$2\,(3 + 3 + 3 + 7 + 7 \text{ perimeter}) \times 114 = 6\,607 \text{ Btu per hr}$$

eat-Losses for Tall Buildings. It is advisable to increase the
osses above the tenth floor by approximately 15% for walls
to the prevailing winds.

d by Persons, Lights, Motors, Machinery, etc. (c) The
emitted by persons is ordinarily not of sufficient importance
account, except in cases of assembly-halls and theaters. The
ces may be made when required:

rest	400 Btu per hour
work	500 Btu per hour

oduced by lights is as follows:

s:

hour equals watts per lamp × number of lamps × 3.415

roducer gas	150 Btu
luminating gas	700 Btu
atural gas	1 000 Btu

rner averages 3 cu ft of gas per hour and a fish-tail burner

lotors and the machinery which they drive, if both are located
vert all of the electrical energy supplied into heat, which is
om if the product being manufactured is not removed until its
same as the room-temperature.

If power is transmitted to the machinery from the outside,
-equivalent of the brake horse-power, d.h.p., supplied is used.
the

$$\text{supplied per hour} = \frac{\text{motor horse-power}}{\text{efficiency of motor}} \times 2\,546$$

case

$$\text{Btu per hour} = \text{d.h.p.} \times 2\,546$$

the Btu equivalent of 1 horse-power hour. In high-powered
ief source of heating and is sometimes sufficient to overheat
n zero weather, thus requiring cooling by ventilation the year

or Estimating the Heat-Loss of Buildings. There is
RULE-OF-THUMB METHODS for estimating the heat-loss H
the heating-surface required when direct radiation is to be

used. These so-called practical rules are intended to be based on aver building-construction and on the ratio of wall and glass-surface to the cubi contents as found in buildings of the class to which they refer. These ru when modified for unusual conditions and applied by engineers of long expe ence in the proportioning and design of heating systems produce satisfact results. They are, however, rapidly being discarded except as rough che on the more refined methods of calculation.

Carpenter's Rule. The following formula, or rule, which has been wid used for many years in this country, was proposed by R. C. Carpenter. It

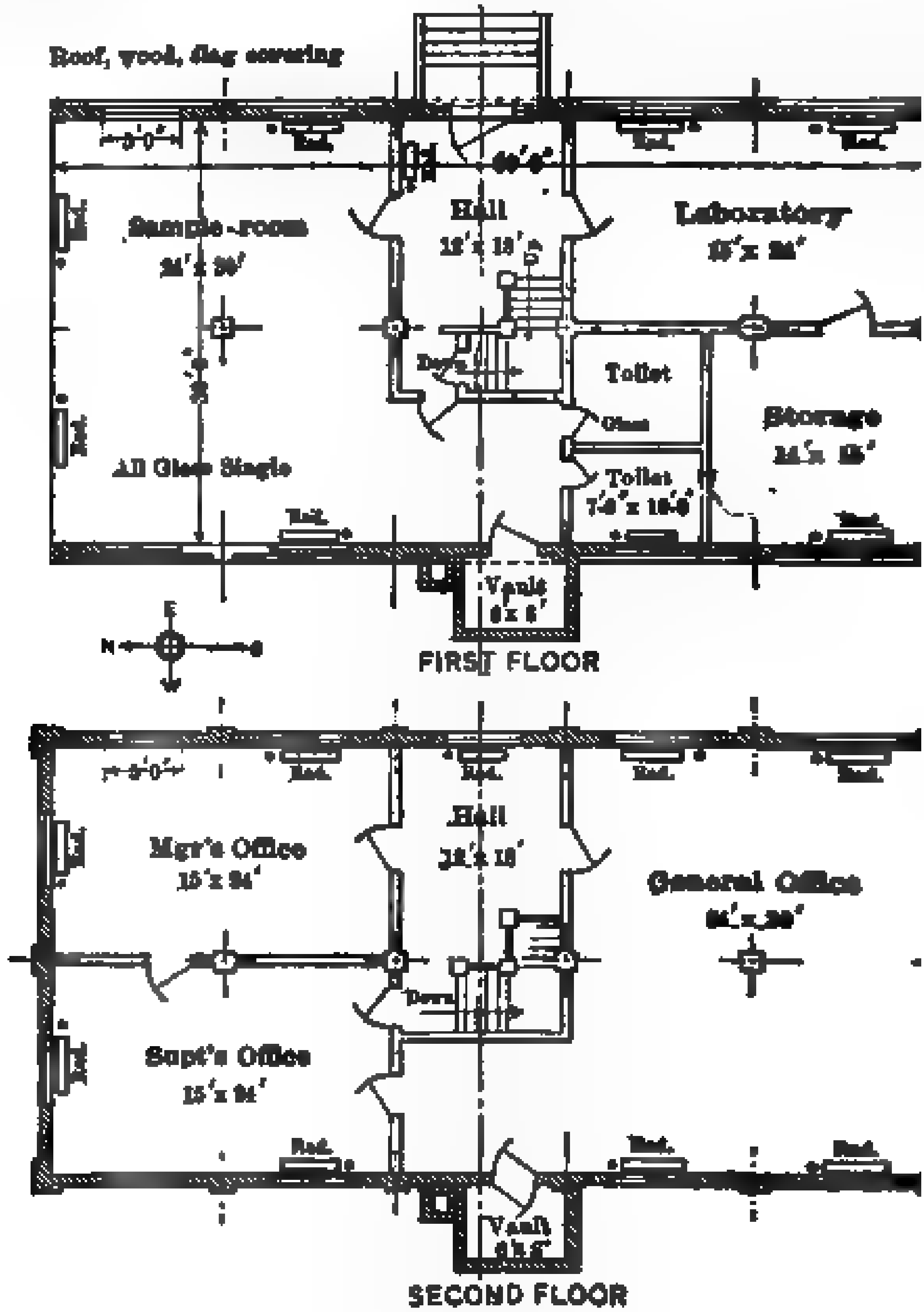


Fig. 9. Floor-plans and Section of Building Explained in Table VII. (See, also, Fig.

not intended to be applied to buildings covered with corrugated sheet steel metal lath and plaster walls, unless the wall-constant is changed to suit the c dition.

By reference to Table IV, it will be noted that a fair average value for t heat-transmission of the usual well-constructed building-wall is approximat

Table VII. Tabulation of Heat-Losses for Building Shown in Fig. 9.

Room-designation	Net volume, cu ft	Net wall-area, sq ft	Floor or ceiling, sq ft	Glass-area, sq ft
1	2	3	4	5
First floor:				
Sample-room.	10 080	852	864	180
Hall........	2 595	99	216	45
Laboratory..	4 320	378	360	90
Office.......	2 520	288	210	60
Toilet.......	900	90	75	30
Second floor:				
Mgr's office..	4 320	393	360	75
Hall........	2 595	119	216	25
Gen'l office..	10 080	852	864	150
Sup't's office..	4 320	393	360	75
Totals....	41 730	3 464	....	730

Room-designation	Transmission-loss, Btu per hour			Infiltration loss, Btu per hour		Total heat-loss, Btu per hour
	Wall-loss 19.7 × col. 3	Floor or ceiling-loss, 13. × col. 4 18.8 × col. 4	Glass-loss, 78.8 × col. 5	Assumed no. air-changes per hour	Infiltration-loss, 1.26 × col. 2 × col. 9	
1	6	7	8	9	10	11
First floor:						
Sample-room.	16 784	11 232	14 184	1	12 700	54 900
Hall........	1 950	2 808	3 556	3	10 809	19 123
Laboratory..	7 446	4 680	7 112	2	10 886	27 358
Office.......	5 674	2 730	4 728	2	6 350	19 482
Toilet.......	1 773	975	2 364	2	2 268	7 380
Second floor:						
Mgr's office..	7 742	6 768	5 910	2	10 886	31 306
Hall........	2 344	4 061	1 970	3	10 809	19 224
Genl. office..	16 784	16 243	11 820	2	25 400	70 247
Sup't's office..	7 742	6 768	5 910	2	10 886	31 306
Totals....	......	......	......	....	100 994	280 326

.25 Btu and for glass 1.0 Btu per degree difference between the inside and outside temperature per hour.

Professor Carpenter states that usually we may, with sufficient accuracy, neglect all inside walls, floors and ceilings and consider only the outside walls.

The estimated number of air-changes per hour, by infiltration, has already been given in Table VI.

Let C = cubical contents of room in cubic feet;
n = number of air-changes per hour (see Table VI);
0.02 = Btu to raise 1 cu ft of entering air 1° F.;
W = net wall-surface in square feet;
G = glass-surface in square feet;
$(t - t_0)$ = temperature-difference between inside and outside;
H = total heat to be supplied per hour in Btu;
$H = (0.02nC + G + \frac{1}{4}W)(t - t_0)$.

Calculating the Heat-Loss of a Building. The following example (Tal VII) will serve to illustrate the method employed in calculating and tabulati the heat-loss of a typical building, the floor-plans and section being shown in F 9. (See, also, Fig. 34.) The heating requirements are for a temperature of 70° in zero weather. The heat-transmission for the outside walls per square fo is taken from Table IV for a temperature-difference of 70°. The heat-lo through the first floor is based on a temperature-difference of 70 − 35 or 35 The heat-transmission per square foot per 1° difference in temperature p hour for 1¼-in wood is 0.37; hence for 35° it is 0.37 × 35 = 13 Btu per hou The heat-loss through the ceiling of the second floor is based on a temperatu difference of 70 − 23 = 47°, 23° being the assumed temperature of the att in zero weather. The heat-transmission per square foot per hour is therefo 47 × 0.40 = 18.8 Btu. The infiltration-loss is, in this example, based on t estimated number of air-changes per hour as indicated in Table VII.

By Carpenter's rule the heat-loss of this building based on two air-chang per hour, is

$$[0.02 \times 2 \times 41\,730 + (3\,464/4) + 730] \times 70 = 228\,564 \text{ Btu per hour}$$

Radiation

Direct Radiation. Steam or hot-water radiators placed in the room to b heated are termed DIRECT RADIATORS or DIRECT RADIATION. Common typ of direct radiators are shown in Figs. 10, 11, 12 and 13.

Indirect Radiation. Radiators used to warm the air passed over them, t heating of the building being accomplished by hot air, are termed INDIRE RADIATORS or INDIRECT RADIATION. (See Figs. 45 and 46.) This type radiation is frequently used for installations in which provision must be ma for ventilation as well as heating, as in the case of schools, public buildings, et Indirect radiation is also used to some extent in high-grade residence-heati where direct radiation may be thought unsightly, particularly for the first flo Direct radiation is ordinarily employed for the floors above the first floor. T principal use of indirect radiators is in connection with the HOT-BLAST SYST of heating, described later, in which a fan is used to circulate the air over t radiator and through the duct system.

Direct-Indirect Radiation. DIRECT-INDIRECT RADIATORS (Fig. 14) a radiators placed in the rooms to be heated and furnished with a cold-air co nection through the outside wall. It serves the purpose of providing te pered-air ventilation.

Materials and Connections of Radiators. Radiators are constructed cast iron, pressed steel or pipe-coils. The sections for one-pipe steam syste are connected only at the bottom. The sections for hot-water radiators a two-pipe steam systems are connected at both top and bottom. The latt is known to the trade as HOT-WATER RADIATION.

adiation. Cast-iron radiators should not be operated above ssure. Standard pipe-coil direct radiation, up to 125 lb.

ators. Radiators are rated according to the square-foot area ıg-surface. Cast-iron and pressed-steel direct radiators are ns. The amount of heating-surface per section of cast-iron various standard heights manufactured is given in Table VIII.

ree-column Radiator **Fig. 11. Peerless Three-column Radiator**

Direct Cast-Iron Radiator. The American Radiator Complaced on the market a new type of direct cast-iron radiator proximately 30% more heating-surface for a given floor-area is is than with other types of direct radiation. The length of and the width 8 in for all heights.

ace per section is as follows:

38-in, 4½ sq ft	34½-in, 4 sq ft	31-in, 3½ sq ft
23-in, 2½ sq ft	19⅓-in, 2 sq ft	

Fig. 12) is largely used in bath-rooms, and also for factory- e width of column-type radiation is objectionable. (See g and dimensions.)

Radiators.* These radiators have been developed in recent st ingeniously fabricated of No. 20 United States standard- ets made into shapes, widths and heights which correspond h the cast-iron column-radiators. Each section is made up of joined by a double-lapped seam and the separate sections single-lapped seams. The pipe-connection is made into a e-iron ring secured to the end-section by rolling the sheet

d by the Pressed Metal Radiator Company, Pittsburgh, Pa.

Table VIII. American Direct Radiators

Heights, widths, lengths and heating-surfaces

Height in inches	45	38	32	26	23	22	20	1
Peerless, single-column, steam and water....	...	3	2½	2	1⅔	...	1½	.
Rococo, single-column, steam and water....	...	3	2½	2	1⅔	..	1½	
Peerless, two-column, steam and water.	5	4	3⅓	2⅔	2⅓	..	2	..
Rococo, two column, steam and water......	5	4	3⅓	2⅔	2⅓	...	2	
Verona, steam and water..........	...	4	3⅓	2⅔	...	..	2	
Peerless, three-column, steam and water....	6	5	4½	3¾	...	3	..	2
Rococo, three-column, steam and water.....	6	5	4½	3¾	...	3	..	2
Peerless, four-column, steam or water......	10	8	6½	5	...	4	..	3
Rococo, four-column, steam or water.......	10	8	6½	5	...	4	.	3
Aetna flue, steam or water............. ..	...	....	..	.	...	...	6	5
Italian flue, steam or water...............	...	7	5¾	4½	...	...	3½	.
Rococo window, steam or water...........	..	...	...	...	...	...	5	.

	16	15	14	13	Length per section in inches	Width of section in inches
Peerless, single-column, steam and water....	...	....	...	...	2½	4½
Rococo, single-column, steam and water....	...	...	...	...	2½	4½
Peerless, two-column, steam and water.....	...	1½*	...	...	2½	7⅝
Rococo, two-column, steam and water......	...	....	...	...	2½	7⅝
Verona, steam and water	...	....	...	...	2½	8
Peerless, three-column, steam and water....	...	....	...	...	2½	9
Rococo, three-column, steam and water....	...	....	...	...	2½	9
Peerless, four-column, steam or water......	...	....	...	...	3	10½
Rococo, four-column, steam or water.......	...	....	...	...	3	10½
Aetna flue, steam or water...............	4⅔	....	4	3⅔	3	12½
Italian flue, steam or water...............	...	....	...	...	3	8½
Rococo window, steam or water...........	3¾	....	...	3	3	12½

* Peerless 15-in in steam only.

The location of the figures in the above columns in line with the names of patt of radiators indicates the heights in which the various patterns are made. The figu themselves represent the amount of heating-surface contained in each section.

To obtain the total length of the radiator, multiply the length per section by the nu ber of sections.

Table IX. American Rococo Wall-Radiators

Ratings and measurements of sections

Section-numbers	Length, in	Width, in	Thickness, in	Thickness (with bracket), in	Heating surface, sq ft
5-A............	16⅝	13 5/16	2⅞	3⅜	5
7-A and 7-B....	21⅞	13 5/16	2⅞	3⅜	7
9-A and 9-B....	29 1/16	13 5/16	2⅞	3⅜	9

er a suitable flange on the inner face of the ring Air-valve made in a similar manner. See Fig. 13 and Table X. These

d Installation of Rococo Wall-radiators in Single Tier on Adjustable Brackets

at in weight and therefore easy to handle and install, and cost nd shipping charges. For the same height, width and area of

13. Presto Single-column Pressed Metal Radiator

these radiators are shorter than cast-iron radiators, being d of $2\frac{1}{4}$ in, center to center of sections.

oil Radiation is largely used in manufacturing establishments ade up of $1\frac{1}{4}$ or $1\frac{1}{2}$-in pipe screwed into cast-iron manifolds 15.

n of Direct Radiation. The unit heat-transmission K, or the by one square foot of direct radiation per hour per degree dif-

Table X. Presto Single-Column Floor or Wall-Radiators for Steam or Wat

Each section is 4½ in wide. Legs spread 5½ in

Number of sections	Length * 1½ in per section	Heating-surface in square feet					
		32 in high	26 in high	23 in high	20 in high	17 in high	14 in high
		2 sq ft per section	1.5 sq ft per section	1.3 sq ft per section	1.1 sq ft per section	0.9 sq ft per section	0.7 sq per sectio
4	6	8	6.0	5.2	4.4	3.6	2.8
5	7½	10	7.5	6.5	5.5	4.5	3.5
6	9	12	9.0	7.8	6.6	5.4	4.2
7	10½	14	10.5	9.1	7.7	6.3	4.9
8	12	16	12.0	10.4	8.8	7.2	5.6
9	13½	18	13.5	11.7	9.9	8.1	6.3
10	15	20	15.0	13.0	11.0	9.0	7.0

* Length of radiator over all, including malleable-iron hubs. Add ¾ in for each bushi
Legs are detachable and can be applied to any section.
These radiators are tapped 1½ in and bushed as specified.

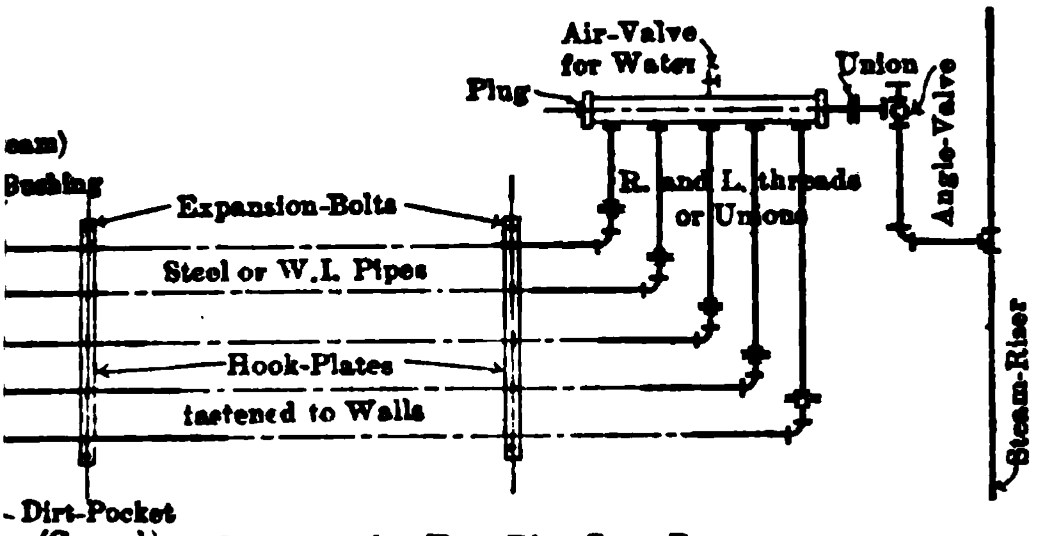
ference between the heating-medium and the temperature of the air in the roo varies somewhat with the type of radiator, height, temperature, etc.

Fig. 14. Direct-indirect Radiator-installation

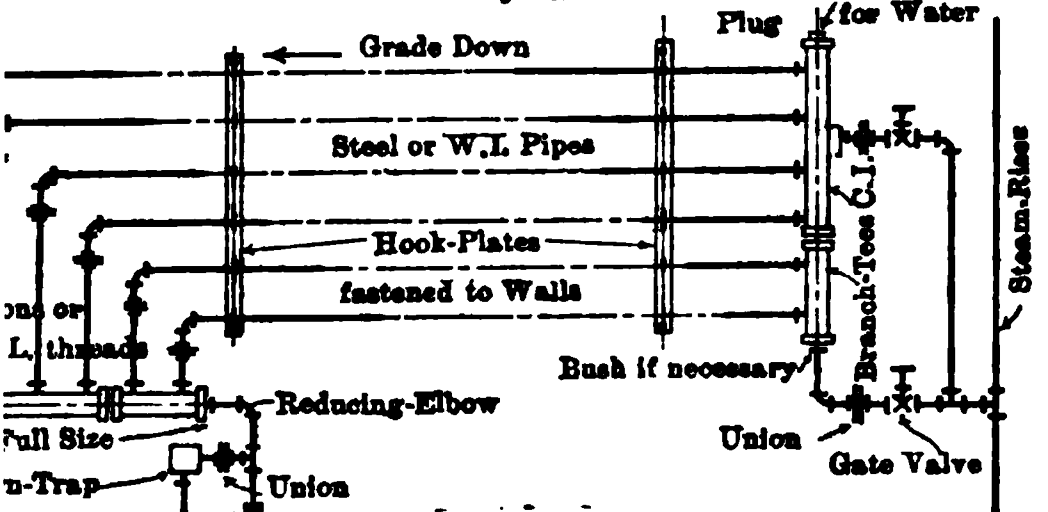
Coefficients of Transmission for Direct Steam-Radiators. Table is based on the average performance of direct steam-radiators standing expo in still air at 70° F. with steam at 220° F., or 2-lb pressure, with a stand

-ence of 150°. In order to apply the coefficients given in -litions other than standard, it is only necessary to know the

Pipe-Coil Radiators and Connections



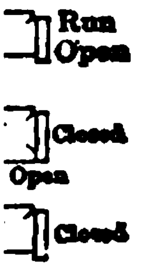
Radiator for Two-Pipe Low-Pressure Steam-System.



Radiator for Two-Pipe Vacuum Steam-System, Divided Surface.

TABLE

1" Branch-Tees 2½" c–c			1¼" Branch-Tees, 3" c–c			1½" Branch-Tees, 3½" c–c			2" Branch-Tees, 4½" c–c		
Runs			Runs			Runs			Runs		
1"–1¼"	1½"	2"	1¼"–1½"	2"	2½"	1½"–2"	2½"	3"	2"	2½"–3"	3½"
No. of Branches			No. of Branches			No. of Branches			No. of Branches		
2 to 9	2 to 16	2 to 16	3 to 16	3 to 16	3 to 16	3 to 12	3 to 12	3 to 12	3 to 10	3 to 10	3 to 10
Inside Diams.			Inside Diams.			Inside Diams.			Inside Diams.		
1¾"	2¾"	2¾"	2½"	2½"	2½"	2¾"	2¾"	2¾"	3½"	3½"	3½"



-ber
1"'H.S.
1"'H.S.
1"'H.S.
1"'H.S.
1"'H.S.

NOTES. All openings in Branch-Tees for circulation are tapped right hand.
Branch-Tees for Box Coils are always tapped left hand in branches and right hand in back inlet.
The run and back openings of Branch-Tees are tapped the same size as branches, unless otherwise ordered.

Fig. 15. Pipe-coil Radiation-data

given increase or decrease in the temperature-range above -tandard range. An examination of test-data so far avail-

able seems to indicate, that this variation is nearly 0.2% per degree above below the standard range of 150°. Thus, if a three-column, 38-in high, dir radiator is to be used in a room kept at 60° F., with steam at 230°, we wo have a temperature-range of 170° or 20° above standard, and the value of would become

$$K = (1.55 + 0.002 \times 20 \times 1.55) = 1.61$$

and each square foot of radiation would give off $1.61 \times 170 = 274$ Btu per

Table XI. Values of K for Direct Radiators

Type of radiator	Height of radiator			
	20 and 22 in	26 in	32 in	38 in
One column	1.95	1.90	1.85	1.80
Two columns	1.80	1.75	1.70	1.65
Three columns	1.70	1.65	1.60	1.55
Four columns	1.60	1.55	1.50	1.45
Flue, 42 sq ft	....	....	....	1.57*
Window	1.85	....	....	....
Wall (horizontal)	1.95	....	....	....
Wall (vertical)	1.90	....	....	....
Pipe-coils	2.00	....	....	....

* Air entering flues at 70° F. and leaving same at 152° F. Allen.

K increases (1) as height of radiator is reduced and (2) as number of columns width of radiator decreases.

Coefficients of Transmission for Direct Hot-Water Radiators. Table may be used for values of K for hot-water radiators of the same type as th listed, but allowance should be made for the lower temperature-range in h water heating. Thus, with a room usually at 70° F., and water at 180° enteri and at 160° leaving the radiator, the temperature-range is only 100°, or 50° l than the standard range. Then for a two-column 26-in high direct radiat the value of K becomes

$$K = (1.75 - 0.002 \times 50 \times 1.75) = 1.58$$

and each square foot of this radiation gives off, $1.58 \times 100 = 158$ Btu per h

Concealed Radiators. The effect of placing a grill in front of a dir radiator, with a cover over the top, reduces the heat-emission by approximat 20%. A clear space between the radiator, wall and enclosure should not less than 2½ in. Concealed radiators are not looked upon with favor from strictly sanitary point of view.

The Usual Assumptions Made for the Heat-Transmission of Dir Radiation is 250 Btu per sq ft per hr for low-pressure steam (2 lb) cast-iron ra ators, and 150 Btu per sq ft per hr for cast-iron hot-water radiators with t water at 180°. The square-foot rating of heating-boilers is based on the abo figures. For more exact values use the data given in Table XI. According a hot-water installation requires 66⅔% more radiation than a low press steam system.

Example. It is required to determine the amount (R) of direct cast-ir radiation, low-pressure steam and hot water, to supply a heat-loss of

$$H = 10\,000 \text{ Btu per hr}$$

tion. For the direct steam system

$$R = H/250 = 40 \text{ sq ft}$$

a direct hot-water system

$$R = H/150 = 66\tfrac{2}{3} \text{ sq ft}$$

ee-column cast-iron radiator, 38 in high, is to be used, the heating-surface h is 5 sq ft per section, it will require $40/5 = 8$ sections for the steam-job, the length of radiator equal to $8 \times 2\frac{1}{2} = 20$ in.

Fuels and Combustion

ification of Fuels. Fuels are generally classified as solid, liquid, and . SOLID FUELS are coal, wood, and wastes. LIQUID FUELS are petroleum products. GASEOUS FUELS are natural and artificial gas.

Fields in the United States. Most of the anthracite is found in beds han 500 sq miles in area located in eastern Pennsylvania. The prin- posit of semibituminous coal is about 300 miles long by 20 miles wide along the eastern edge of the Northern Appalachian field. The bitumi- ls extend from this deposit westward. A little graphitic coal is found Island.

osition of Coal. The uncombined carbon in coal is known as FIXED Some of the carbon-constituent is combined with hydrogen, and this, with other gaseous substances driven off by the application of heat, t portion of the coal known as the VOLATILE MATTER. The fixed carbon olatile matter constitute the COMBUSTIBLE. The oxygen and nitrogen l in the volatile matter are not combustible, but custom has applied to that portion of the coal which is dry and free from ash, thus includ- ygen and nitrogen in the combustible.

ication of Coals. Coals may be classified according to the percentages arbon and volatile matter contained in the combustible.

Table XII. Classification of Coals (Kent)

e of coal	Percentages of combustible		Btu per pound of combustible
	Fixed carbon	Volatile matter	
ite........	97.0 to 92.5	3.0 to 7.5	14 600 to 14 800
hracite	92.5 to 87.5	7.5 to 12.5	14 700 to 15 500
minous ...	87.5 to 75.0	12.5 to 25.0	15 500 to 16 000
ous. East...	75.0 to 60.0	25.0 to 40.0	14 800 to 15 300
ous West...	65.0 to 50.0	35.0 to 50.0	13 500 to 14 800
....	50.0 and under	50.0 and over	11 000 to 13 500

etric Determinations. The only accurate and reliable way to deter- eating-value of a fuel is to do so experimentally with a calorimeter. uels, the BOMB-CALORIMETER is the most practical. The various types ket include the Mahler, the Hempel, the Atwater and the Emerson. ist essentially of a tight vessel containing a weighed sample and oxygen sure. This receptacle is placed within another vessel containing a ght of water and surrounded by heat-insulating material to minimize

radiation. The sample is EXPLODED electrically, and the heat absorbed by surrounding water is determined by means of a very accurate thermome reading hundredths of a degree. Correction has to be made for the heat sorbed by the instrument itself, and for radiation.

For a complete description of calorimeters and their use, see Carpenter Diederichs' Experimental Engineering.

Calorific Value by Formula. The following expression, known as Du Lo FORMALA for heating-value per pound of coal, can be used if the ultim chemical analysis of the fuel is known:

$$F = 14\,600\,C + 62\,000\,(H - \tfrac{1}{8}O) + 4\,000\,S$$

where C, H, O, and S represent the proportionate parts of each element per of fuel, and F denotes the heat-value in Btu per pound due to combusti This formula does not apply when the fuel contains carbon monoxide, CO, can be made to apply by adding a term, 10 150 C, in which C is the proportion part of carbon burned to the monoxide.

Example. The application of the formula to a coal of ultimate analysis here given follows:

Analysis (based on fuel as received)

C	74.79%
H	4.98
O	6.42
N	1.20
S	3.24
H_2O	1.55
Ash	7.82
	100.00%

Then by Du Long's formula, 14 600 × 0.7479 + 62 000 (0.0498 − 0.0642 + 4 000 × 0.0324 = 13 650 Btu per 1 lb of coal.

A bomb-calorimeter test showed 13 480 Btu for this coal. The formula fa to allow for evaporating and superheating the moisture present in the fuel.

Combustion of Fuel. Combustion, as used in steam-engineering, signifies rapid chemical combination between oxygen, and the carbon, hydrogen, a sulphur composing the various fuels. This combination takes place usually

Table XIII. Theoretical Amount of Air Required for Combustion

Fuel	Composition by weight			Lb of air per lb of fuel
	% C	% H	% O	
Wood-charcoal	93.0	....	....	11.16
Peat-charcoal	80.0	....	....	9.6
Coke	94.0	....	....	10.8
Anthracite coal	91.5	3.5	2.6	11.7
Bituminous coal, dry	87.0	5.0	4.0	11.6
Lignite	70.0	5.0	20.0	8.9
Peat, dry	58.0	6.0	31.0	7.6[illegible]
Wood, dry	50.0	6.0	43.5	6.0[illegible]
Mineral oil	85.0	13.0	1.0	14.3[illegible]

temperature with the evolution of light and heat. The su
; with the oxygen is known as the COMBUSTIBLE, and if it
d or oxidized the combustion is PERFECT, that is, no more
up by the products of the reaction. The combustion is
plete when carbon burns to form carbon monoxide, CO, i
e, CO_2, since the former may be further burned to form car
essary oxygen is supplied. It is necessary to provide for an
burning coal under either natural or forced draft, amountin
50 to 100% of the net calculated amount, or about 18 to
Less air results in IMPERFECT COMBUSTION and smoke, while
and setting and carries away a large percentage of the he

IV. Weight and Calorific Value of Various Gases at 32 De heit and Atmospheric Pressure, with Theoretical Amount Required for Combustion

Gas	Symbol	Cubic feet of gas per pound	Btu	
			Per pound	Per cubic foot
gen	H	178.0	62 000	348
n monoxide	CO	12.81	4 380	342
ne	CH_4	22.4	23 842	1 065
e	C_2H_6	12.0	22 400	1 865
ne	C_2H_4	12.8	21 430	1 675
ene	C_2H_2	13.79	21 430	1 555

Storage. Space for fuel-storage must be based on fuel-con
s estimated under Fuel-Consumption, page 1278, and in
it is customary to proportion the storage-space on the b
area per ton, the storage-space being made ample to h
supply.
llowing volumes per ton of 2240 lb of coal are given for
pace: bituminous coal, 41 to 45 cu ft, and may run as hig
e coal, 34 to 41 cu ft; charcoal, 123 cu ft; coke, 70.9 cu
based on fuel broken down ready for market. Also
lb and 1 bushel soft coal = 76 lb.

Steam-Heating Boilers and Hot-Water Heaters

res, Attention, and Materials. Heating-boilers usually
ER PRESSURE than do power-boilers, and in most cases r
The steam-boilers are usually designed to operate on f
SSURE, and the WATER-BOILERS or hot-water heaters ar
hydrostatic head in excess of 100 FT when in operation.
these boilers is of such an INTERMITTENT CHARACTER t
heating-load for comparatively long periods without
y range from 6 to 10 hrs and in consequence the comb
elatively large grates and fire-pots are necessary. T
for constructing heating-boilers are CAST IRON, espe
es, although boilers of nearly 100 equivalent steam
Rating of Heating-Boilers) are made of this same

STEEL OR WROUGHT IRON, which are more generally used in the larger sizes. T government departments usually specify steel heating-boilers, and they are u extensively in office and loft-buildings as well.

Boiler Heating-Surface. The CAPACITY of any boiler or water-heater depe on the amount of, and the temperatures on the opposite sides of, the he transmitting surfaces in contact with the water in the boiler on one side, and t fire or hot gases on the other. It is most important that a rapid circulation water and the hot gases shall take place over these surfaces, and preferably opposite directions. Two kinds of surface are distinguished in boiler-practi and known as direct and indirect surface. DIRECT SURFACE is that on whi the fire shines, and INDIRECT that in contact with the flue-gases only. All su surface must have water on the opposite side. In some boilers the hot ga are allowed to come in contact with the boiler-surface above the water-li so that there is only steam in contact with this surface on the inner side. Su surface is known as SUPERHEATING-SURFACE in order to distinguish it fr ordinary heating-surface. Direct surface is the more valuable of the two, p square foot, as it is usually subjected to a higher temperature, and furtherm because the intensity of radiation from an incandescent surface appears to va as some power of the temperature of that surface, either the third or fourt

Equivalent Evaporation. The equivalent evaporation of a boiler is t pounds of water the boiler would evaporate per pound of coal burned if received the feed-water at 212°, and evaporated it into steam at this same te perature and pressure, so that the evaporation would take place FROM AND 212° F. In practice the feed-water is usually below this temperature a evaporation actually takes place at some higher temperatrue than 212°. He to find the EQUIVALENT EVAPORATION it is always necessary to make use of t following relation:

$$E = \frac{(x_2r_2 + q_2 - q_1)}{971.7} \times P$$

where the fractional part of the expression is known as the FACTOR OF EV ORATION; so that

$$E = \text{factor of evaporation} \times P$$

E = equivalent evaporation from and at 212° F., in pounds;
x_2 = quality of steam as actually evaporated;
r_2 = latent heat of steam as actually evaporated;
q_2 = heat of the liquid as actually evaporated;
q_1 = heat of the liquid as actually fed to boiler;
P = actual evaporation in pounds per pound of fuel burned;
971.7 = latent heat of steam at 212° F.

Boiler Horse-Power. A boiler horse-power is the energy required to ev orate 34.5 lb of water at 212° F. into DRY STEAM of 212° F., or

$$971.7 \times 34.5 = 33\,524 \text{ Btu}$$

The HORSE-POWER RATING of a boiler is always measured in terms of the equi lent evaporation. Thus, if we divide the EQUIVLAENT EVAPORATION of a bo by 34.5 we get the boiler horse-power developed.

Boiler-Efficiencies. Heating-boilers, operated at their rated capacity, show an EFFICIENCY of from 55 to 65%. This efficiency is the ratio of h absorbed per pound of dry coal by the water and steam in the boiler to the ac heat-value of one pound of the coal, and is the COMBINED EFFICIENCY of boiler and furnace.

les of Combustion for Heating-Boilers. Combustion-rates for varying of grates are given in Table XV:

Table XV. Combustion-Rates

Grate-areas	Coal per square foot per hour, in pounds	Remarks
sq ft or less (small),	5	A variation of 10% up or down from these rates is perfectly safe. The higher values are for full-sized chimneys with lined flues and the lower for unlined flues or long breeching-connections.
–10 sq ft medium),	5.7	
ı ft or larger (large),	6.6	
to 8 sq ft	4	(Am. Soc. H. and V. E. Com. 1909.) Rates of combustion reported for anthracite coal, as fired in internally fired heating-boilers. See Transactions for further details.
to 18 sq ft	6	
to 30 sq ft	10	

ıg of Heating-Boilers. Standard Conditions. It is the general cus- American manufacturers of heating-boilers to rate their boilers in terms umber of square feet of standard direct cast-iron radiating-surface which er is capable of supplying under the following conditions:

team boilers; steam-pressure 2-lb gauge at boiler.

'ot-water boilers; water-temperatures: 180° F. leaving, and 160° F. boiler.

uel; anthracite coal of stove-size.

ATE OF COMBUSTION, or amount of coal necessary per hour for the boiler op its rating has, until recently, seldom been given; and the method of ing the rating has varied with different makers and is seldom stated. r, it is possible for a boiler to be placed on the market and assigned a ating although such rating has never been actually checked by test. ore becomes most important to not only establish STANDARD CONDITIONS NG-TESTS, but to require the manufacturer to be in a position to pro- tified test-sheets of such tests for his line of boilers. The STANDARD NS under which a boiler should be tested to develop its rating are gen- derstood by the manufacturers at the present time to be as follows:

essure, temperature and fuel as stated above.

el-capacity to be sufficient to carry the boiler from 6 to 8 hr on one id leave 20% reserve for igniting fresh charge.

aft of sufficient intensity to burn the fuel at the required rate. A not less than 40 ft in height is recommended.

ch square foot of direct cast-iron radiation has a transmission-value of and 150 Btu per hour for steam and water-radiators respectively.

condensation from steam-radiators returns to the boiler at the same ure as the steam, or without loss of heat, so that the boiler simply sup- atent heat of evaporation at 2 lb pressure, or 967 Btu per lb evaporated.

water from hot-water radiators returns to the boiler at 160°, allowing in the radiators, so that there is no loss in temperature allowed in -main.

able heat-allowance must be made for all connecting piping and boiler- ıd such surface must be figured as radiating-surface or its equivalent.

A general rule is to add, for an ordinary installation, about 50% of the sq ft radiation installed, in calculating the total load on the boiler, with anthrac fuel and 65% with bituminous fuel, to allow for radiation-loss of piping and boi and the additional tax on the boiler due to starting up with cold radiation.

Equivalent Boiler Horse-Power Rating of Heating-Boilers. T capacities of heating-boilers may be stated in boiler horse-power, and the equiv lent of same in square feet of standard radiation may be easily determined follows:

Since 1 boiler horse-power is equal to 34.5 lb of water evaporated per ho from and at 212° F., the boiler must deliver

$$34.5 \times 971.7 \text{ (latent heat at } 212° \text{ F.)} = 33\,524 \text{ Btu per hr}$$

Now since 1 sq ft of standard cast-iron steam-radiation transmits 250 B per hour,

$$1 \text{ boiler horse-power} = 33\,524/250 = 134.1 \text{ sq ft of this radiation, or}$$

$$1 \text{ sq ft of direct cast-iron steam-radiation} = 0.00756 \text{ boiler horse-power}$$

It also follows that the equivalent boiler horse-power rating of a hot-wa heater is

$$33\,524/150 = 223.5 \text{ sq ft of direct cast-iron hot-water radiation, or}$$

$$1 \text{ sq ft of direct cast-iron hot-water radiation} = 0.00447 \text{ boiler horse-powe}$$

Grate-Surface. It is always advisable to check THE GRATE-AREA REQUI for heating-boilers, especially if the total heat-loss to be supplied by the bo is known. This total heat-loss must include not only the calculated loss, d to transmission through walls and glass, for which the radiation is proportion but also about 50% additional for heat-losses from the piping system, boi etc. So that, if H is the building-loss in Btu, $1.5\,H$ = total Btu-loss.

Then

$$G = 1.5H/(C \times F \times E)$$

where C = rate of combustion in pounds of dry coal per square foot of gra area per hour, F = calorific value of fuel in Btu per pound of dry coal (12 0 is the usual assumption for anthracite coal), and E = the combined efficiency boiler and grate (60% is the usual assumption). G is in sq ft and the boi selected should have not less than this grate-area. Special attention is cal to the distinction between GRATE-AREA and FIRE-BOX or FUEL-POT AREA explained below under Depth of Fuel-Pot.

Depth of Fuel-Pot. The average of the fire-box area is usually somew larger than the grate-area in sectional boilers, while it may be less than grate-area in certain types of round boilers. In any event the capacity of fire-box or fuel-pot from grate to middle of fire-door should always be s ficient to hold all the coal required for an 8-hr firing-period, plus at least a reserve to be used for igniting a fresh charge.

The following method is used to determine the depth of pot or the firing-pe as the case may be. Let G = grate-area in sq ft, C = rate of combustion, A average area of fire-pot, h = firing-period in hours, W = weight of fuel per c (50 lb for anthracite and 40 lb for bituminous), D = depth of fuel-bed in Then $(GC\,h)$ + 20% (allowance to ignite fresh charge) = total weight of charge; also, AWD = total weight of one charge. Hence

$$D = 1.2GCh/AW, \text{ or } h = AWD/1.2GC$$

oted above D is measured from grate to center of fire-door, which varies 3 × 14 in in small, to 11 × 19 in in large boilers. This formula allows greater bulk of soft coal.

ple. Given a boiler with grate-area of 8 sq ft, average area fire-pot height to center of fire-door = 18 in, rate of combustion = 6 lb per sq ft e for anthracite coal. Required the number of hours this boiler will ts load on one charging.

ion.

$$h = (9 \times 50 \times 1.5)/(1.2 \times 8 \times 6) = 11.7 \text{ hours}$$

ts of Fuels on Ratings. All ratings are based on ANTHRACITE COAL OF SIZE unless otherwise stated. In case bituminous coal is used and the s selected by catalogue-rating, a boiler with fire-pot having at least 25% capacity should be selected, for the same weight of coal occupies 25% ace. With SOFT COAL additional heating-surface is also required as the lation of soot from such coal renders the heating-surfaces less effective hen hard coal is used. Boilers for PEA-COAL should also have a larger than those for stove or furnace-coal. The SMALL SIZES OF ANTHRACITE far more ash than the larger sizes, and hence have a greater bulk for the ating effect; so that larger fuel-pots for the same capacity are required. PERIODS, differing from the one on which the boiler is rated, will also affect holding capacity. For example, if it is required to operate a certain line s designed for an 8-hr period on a 12-hr basis, at least 50% greater fuel-capacity will be necessary and a larger boiler must be selected, as shown ormula already given for the depth of the fuel-pot.

alent Rating for Conditions Other than Standard. If often happens load connected to a steam-or hot-water boiler may not be operated under dard conditions previously assumed as a basis of rating. In this case ratings cannot be used until the EQUIVALENT VALUE of this load in square feet of standard cast-iron radiation has been determined. llowing relations show a method for finding such equivalent values: = sq ft standard cast-iron radiation = 250 Btu per sq ft for steam, and per sq ft for water. Also let

r = actual sq ft of radiation to be supplied;
K = coefficient of transmission for this radiation;
or t_w = temperature of steam or average temperature of hot water in the radiator;
t_a = temperature of air surrounding radiator;
$t_a)$ = radiation-factor or Btu given off per sq ft per hr;

$$R_s = r_1 \times K_1(t_s - t_a)/250, \text{ and } R_w = r_2 \times K_2(t_w - t_a)/150$$

e. (Steam-heating.) Required the size of boiler (rating in sq ft of cast-iron radiation) to supply 1 000 sq ft of direct pipe-coil radiation. essure = 5-lb gauge. Air = 65° F. K (by test) = 2.42 Btu. From les, t_s = 227.14, R = 1 000 × 2.42(227.14 − 65)/250 = 1 000 × 52.14)/250 = 1 570 sq ft. To this add 50% for pipe and boiler- and the additional tax for starting up with cold radiation, or, 1.5 × 355 sq ft, or practically a 2 400-sq-ft-capacity boiler will be required. should be checked by calculation previously given to ascertain size.

$$G = 1.5H/(C \times F \times E)$$

Example. (Water-heating.) Let Q = total number of gal of water to l heated in h hours.

$$W = (8\tfrac{1}{3} \times Q)/h = \text{weight of water to be heated per hour}$$

$$t_1 = \text{initial temperature of water, } t_2 = \text{ final temperature of water}$$

Then $W(t_2 - t_1)$ = Btu to be supplied per hour. Hence $W(t_2 - t_1)/150$ = ho water-heater rating required. $W(t_2 - t_1)/250$ = steam-boiler rating require

Example. A swimming pool contains 50 000 gal of water, and this water heated by being passed through a hot-water heater in four hours. Enterin temperature = 50° F. and final temperature = 75° F. Hot-water radiati reduced to equivalent standard value = $[(50\,000 \times 8\tfrac{1}{3})/(4 \times 150)]$ $(75 - 50)$ = 17 350 sq ft = rating of hot-water heater, to which must be add 50% for losses from piping, etc.

Fuel-Consumption. The ESTIMATED FUEL-CONSUMPTION FOR HEATIN BOILERS per heating-season may be based on grate-areas, square feet of radiati installed, or cubic contents of building to be heated. The United States Treasu Department allows 5 tons of coal per sq ft of grate-area per season of 240 day or 1 lb of coal per cu ft of contents of building for the same period. Th applies to government buildings. The district steam-heating companies estima 500 lb of steam per sq ft of direct steam-radiation per season, which is practical the same as 70 lb of coal of good quality. This is approximately equivalent assuming that one-third of the radiation installed is in operation continuously f 240 days. In other words, the coal required for a heating-season is about on third the quantity that would be used if all the radiation were in constant u every hour of the day and night. The amount of coal for maximum conditio is determined as follows:

Since each foot of direct steam-radiation or its equivalent will give off 2 Btu per hour under conditions of 2 lb (220°) pressure at boiler, and 70° air su rounding the direct radiators (the piping on the average job may be rough taken as 25% of the direct radiation); and since for approximation we m assume 8 000 Btu per pound of anthracite coal burned; we can readily estima the amount of coal per hour if R = amount of direct radiation in square fe

$$(1.25 \times R \times 2.50)/8\,000 = C = \text{coal per hour in pounds}$$

In a heating-season of 7 months or 210 days of 24 hours each, there would burned under maximum conditions during the entire period

$$(1.25 \times R \times 250 \times 210 \times 24)/(8\,000 \times 2\,000) = 0.0984R \text{ tons of coal}$$

the actual consumption being about one-third of the maximum possible, 0.0328 R tons of coal for the heating-season. For hot-water heating the fu consumption for the entire season is approximately 0.0197 R tons.

Types of Heating-Boilers. Cast-iron steam-heating boilers are designed be operated at a maximum pressure of 15 lb per sq in, and the sections are test by the manufacturer to about 100 lb per sq in, hydrostatic pressure. Cast-ir boilers are constructed of sections, which are connected by means of nipples either the push or screw-type. The sections are held in place by means of lo bolts. Round-type boilers have horizontal sections surrounding the fire-p and in the sectional type the sections are placed vertically. (See Figs. 16, 1 and 18.) The maximum size of round-type boilers manufactured is rated about 1 400 ft. Sectional boilers are obtainable up to a 10 000-sq-ft-rati See manufacturers' catalogues for capacities, dimensions, etc.)

ikeless or Down-Draft Cast-Iron Boilers. Boilers having a water-grate w being made for use with free-burning soft coal, where local smoke-nces would not permit the use of such fuel on ordinary grates.

Fig. 16. Sectional Type of Cast-Iron Boiler

17. Trimmings and Connections for Sectional Steam-heating-boiler

Selection of Cast-Iron Boilers. The selection of cast-iron boilers sho not be influenced too largely by considerations of price, and the ease w which they may be carried into a building where structural conditions interf with the introduction of a steel boiler. many cases the character of the service attendance, or both, especially in gove ment and other public building work, m be such that steel equipment, which is c able of withstanding more abuse, should used. This is particularly true when t returns are handled by a pump. If ca iron boilers are to be installed, the gra area necessary should be carefully comput as already indicated, using an average ra of combustion, and a fuel-pot depth bas on the firing-period required. The Unit States Treasury Department selects ca iron boilers by proportioning them to ca 25% more radiation than actually instal if anthracite coal is used, and 35% more bituminous coal is used. In addition this, suitable allowance must be made mains and other piping, and in most ca two boilers are installed, each capable supplying two-thirds of the radiation order to provide for units which can operated with a high-load factor, and a act as a reserve for each other in case o breakdown.

Fig. 18. Section of Round-type Boiler

Steel Heating-Boilers. There are two general types of all-steel boilers u for heating work, the FIRE-BOX TYPE and the RETURN TUBULAR TYPE.

In the fire-box type the grate and combustion-chamber are surrounded by extension of the steel shell which is water-jacketted. The products of co bustion pass directly through the tubes to the smoke-flue located in the re In the return tubular type, the boiler consists of a shell with tubes set i brick setting, the grate and combustion-chamber being directly under t front portion of the shell. The products of combustion in this case pass und and around the shell to the rear of the boiler, and then through the tubes the front into the smoke-box.

Fire-box type boilers may be obtained in capacities ranging from 500 to 13 0 sq ft of direct radiation. The most common of these boilers are the Dunni Gorton, and Kewanee. Detailed information as to capacities, dimensions, e may be obtained from the makers' catalogues. As usually constructed, th boilers are designed for a working pressure of 60 lb per sq in and are so insu by the boiler-insurance companies. This type may be obtained with or with (portable type) brick-setting. The return tubular boiler is erected wit brick setting and as ordinarily constructed, is designed for a working press of 100 lb per sq in, but may be obtained for a working pressure of 150 lb per s if desired. It is primarily a power-type boiler, but is commonly used in c junction with large heating systems having 10 000 sq ft or more of direct ra tion. These boilers are rated on a basis of 10 sq ft of boiler heating-surface boiler horse-power. A special design of setting is required for smokeless c bustion when bituminous coal is to be used as fuel The so-called stand setting should not be used in this connection. (See Boilers and Rules for C

ion in Mechanical Equipment of Buildings, Vol. II, by Harding and d.)

mneys for Heating-Boilers. (See also, under Chimneys, page 1364.) er to produce an intensity of draft sufficient to properly operate low-e heating-boilers, hot-water boilers, and hot-air furnaces up to their apacity, the chimney should not be less than 40 ft in height, measured ie grate. No flue should be less than 8 × 8 in. The failure of many -installations may be traced to insufficient draft to burn the fuel at the quired to run the boiler or furnace to rated capacity. The tempera-flue-gases leaving the boiler should range between 400° and 500° F. when paratus is worked at its rated capacity. The chimney should be so with reference to any higher buildings nearby that wind-currents will n eddies and force the air downward in the shaft, as shown in Fig. 19. ie should run as nearly as possible from the base top outlet. The outlet ot be capped so that its less than the area of the he flue should have no into it other than the moke-pipe. Sharp bends ets in the flue often reduce and choke the draft, and must be free of any which prevents the full the passage of smoke. If the flue is made of tile, the joints must be iented, or all space between the tile and brickwork filled in tightly. ust be no open crevices into the flue where the tile sections meet, other- draft will be checked. If the flue is made of brick, the stack should side walls at least 8 in thick to insure safety. The inside joints should truck, and each course should be well bedded and free from surplus t the joints. The exposed bricks at the top of a brick chimney should

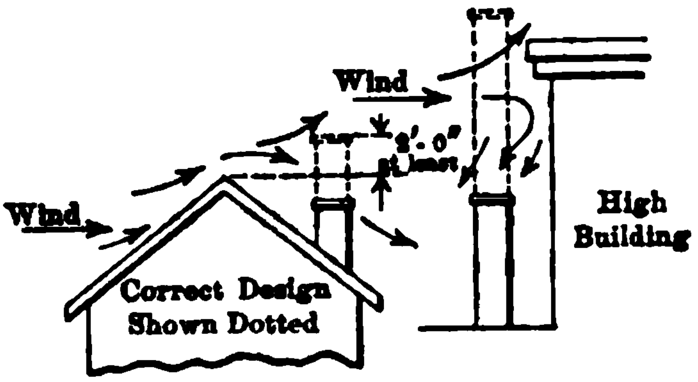


Fig. 19. Relation of Height of Chimney to Draft

Table XVI. Fire-Clay Flue-Linings

Robinson Clay Product Co., Akron, Ohio

Rectangular			Round	
inal e,	Actual size outside, in	Actual size inside, in	Inside diameter, in	Outside diameter, in
8½	4¾ × 8⅝	3¼ × 7	6	7½
13	4¾ × 13¼	3 1/16 × 11¾	7	8½
18	4½ × 17	3⅞ × 15½	8	9
12	6 × 12	4½ × 10½	9	10½
7	7¼ × 7¼	5¾ × 5¾	10	12
8½	8½ × 8½	7¼ × 7¼	12	14
13	8½ × 13	6⅞ × 11⅝	15	17⅛
18	8½ × 18	6½ × 16	18	20⅞
13	13 × 13	11¼ × 11¼	20	23
18	13 × 18	10¾ × 15¾	24	27
18	18 × 18	15½ × 15½	30	35

be laid in cement mortar to prevent the acid fumes and rain from cutting the joints. This will happen if lime mortar is used. The most desirable lo tion for a chimney is near the center of the building, as all walls are then k warm. If there is a soot-pocket in the flue below the smoke-pipe opening, clean-out door should always be tightly closed. If this soot-pocket has ot openings into it from fireplaces or other connections, these openings check draft and prevent the best results. The smoke-pipe should not extend into flue beyond the inside surface of the latter. If it does extend beyond, its cuts down the area of the flue. The joints, where the smoke-pipe fits smoke-hood of the boiler, or where the pipe enters the chimney, should be m tight with boiler-putty or asbestos cement. Fire-clay flue-linings are used the best practice for small and medium-sized flues. Rectangular flue-lini are rated by outside dimensions, and round linings by inside dimensions.

Flues for Kitchen Ranges and Fireplaces. (See also, under Chimn page 1364.) For a kitchen range an 8½ by 8½-in tile flue is ordinarily suffici but an 8½ by 13-in is better. For fireplaces the sectional area of the flue burning wood or bituminous coal should be from 1/10 to 1/8 the area of the f place-opening for a rectangular flue, and 1/12 for a circular flue. For burn anthracite coal the areas may be reduced to 1/12 and 1/16 respectively.

Selection of Chimney-Flues. (See also, under Chimneys, page 1364.) T selection of chimney-flues for heating-boilers must depend upon the judgmen the heating-engineer, but it is believed that Table XVII, by R. C. Carpenter, very much assist the engineer in selecting flues. It is necessary that AREA HEIGHT, THICKNESS OF WALLS, GENERAL STRUCTURE, and the POSITION OF TOP OUTLET with reference to the building and other buildings near by should carefully noted and observed in the selecting or building of a flue. The fig given under the varying heights of chimneys are diameter-measurement inches, or, the side of a square, the theory being that the spirally ascen column of smoke and gases will make a 12 by 12-in flue no more effectiv practical working-area than a twelve-inch round flue. Rectangular shapes be used if the area is equal and the difference in width and breadth is extreme. A maximum ratio of 2 : 1 for the internal dimensions should not exceeded.

Table XVII. Chimneys for Steam and Hot-Water Boilers

Direct radiation		Height of chimney-flue				
Steam, sq ft	Water, sq ft	30 ft	40 ft	50 ft	60 ft	80 ft
250	375	7.0	6.7	6.4	6.2	6.[illegible]
500	750	9.2	8.8	8.2	8.0	7.[illegible]
750	1 150	10.8	10.2	9.6	9.3	8.[illegible]
1 000	1 500	12.0	11.4	10.8	10.5	10.[illegible]
1 500	2 250	14.4	13.4	12.8	12.4	11.[illegible]
2 000	3 000	16.3	15.2	14.5	14.0	13.[illegible]
3 000	4 500	18.5	18.2	17.2	16.6	15.[illegible]
4 000	6 000	22.2	20.8	19.6	19.0	17.[illegible]
5 000	7 500	24.6	23.0	21.6	21.0	19[illegible]
6 000	9 000	26.8	25.0	23.4	22.8	21[illegible]
7 000	10 500	28.8	27.0	25.5	24.4	23[illegible]
8 000	12 000	30.6	28.6	26.8	26.0	24[illegible]
9 000	13 500	32.4	30.4	28.4	27.4	25[illegible]
10 000	15 000	34.0	32.0	30.0	28.6	27.[illegible]

te-Areas and Stack-Dimensions. For return tubular S. Thompson gives the following rules for grate-areas and

. radiation in building; B. H. S. = heating-surface in boiler; ; (all in sq ft)

7 for steam; R ÷ 11 for water. G = B. H. S. ÷ 25 (anthra-oal); G = B. H. S. ÷ 30 to B. H. S. ÷ 35 (bituminous coal, B. H. S. ÷ 45 (lower grate of down-draft furnace).

tack, ft. A = area of grate, sq ft. S = area of stack, sq ft.

(anthracite coal, lump coal, oil, and gas).

) + $\sqrt{H}$ (bituminous and small anthracite).

pea, or rice coal, tube-area must be not less than ⅛ grate, and n stack.

h down-draft furnace, tube-area must be not less than ⅙ of always larger than stack.

th of tube must not exceed 48 diameters.

gth of boilers, 54-in diameter and under, must not exceed 3 54-in, 2½ diameters.

number of feet in length, are not used.

ll Buildings are special cases and may be designed by methods ign of chimneys for power-boilers. (See Power Plants and r Harding and Willard. See also, List of Tall Brick Chimneys,

Direct Steam Heating

Direct Steam Heating in Use. Systems for heating with direct are broadly divided into two general classes, known as: (1) LATING SYSTEMS, and (2) MECHANICAL CIRCULATING SYSTEMS. ing characteristic is the manner in which the water of condensa-diators is returned to the boiler. In the first type the condensate er by gravity, due entirely to the STATIC HEAD existing in the e sytem is a closed circuit. The steam-pressure existing in the and radiators is the same, except for friction-pressure losses due team to the heating-surfaces. In the second type the condensate turn to a receiver or feed-water heater and is then forced into the MP, or RETURN-TRAPS, or both. This is not a closed system, and . the boiler may be much higher than that in the mains and radi-ceiver is usually vented to the atmosphere, and in the case of ns an additional pump is attached directly to the returns and scharge the condensation into the receiver or heater. Gravity cir-ns are further divided into the ONE-PIPE SYSTEM and the TWO-PIPE basement-mains supplying risers to the various floors above (Figs.), or with overhead mains supplying drop-risers to the floors below. system the steam and water of condensation in the risers flow in ction, so that less friction is produced as countercurrents do not aller pipe-sizes may be used. The overhead system is very com-n of as the MILL'S SYSTEM.

Gravity Systems. The ONE-PIPE CIRCUIT SYSTEM (Fig. 20) with ins is probably the simplest, and most common gravity system in eam-main rises close to the basement-ceiling, just above the boiler, des down uniformly from this high point with a fall of 1 or ¼ in in n the last radiator has been served the main drops below the boiler

water-line and its size is reduced, as on the run back to the boiler it carries on condensation and is known as a WET RETURN. This return may be run above the boiler water-line if necessary, and is then called a DRY RETURN. Return-mai are graded 1 in in 30 ft in gravity work. In either case an automatic air-val

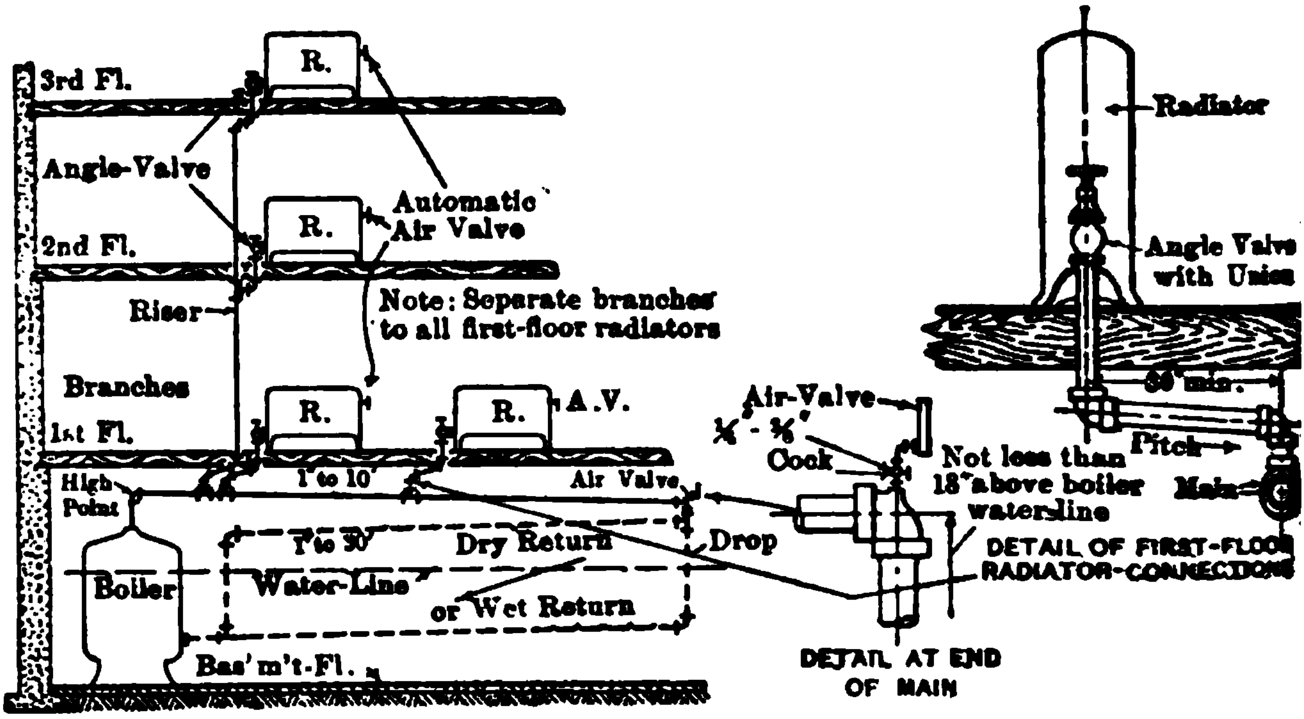


Fig. 20. Low-pressure Gravity System. One-pipe Basement-main

must be installed on the end of the main at the drop, as shown, to vent the sa when air collects in the piping. The elevation of the end of the steam-main wi respect to the boiler water-line must be carefully determined, in order that wa may not back up from the boiler and flood the main, including the air-valve a

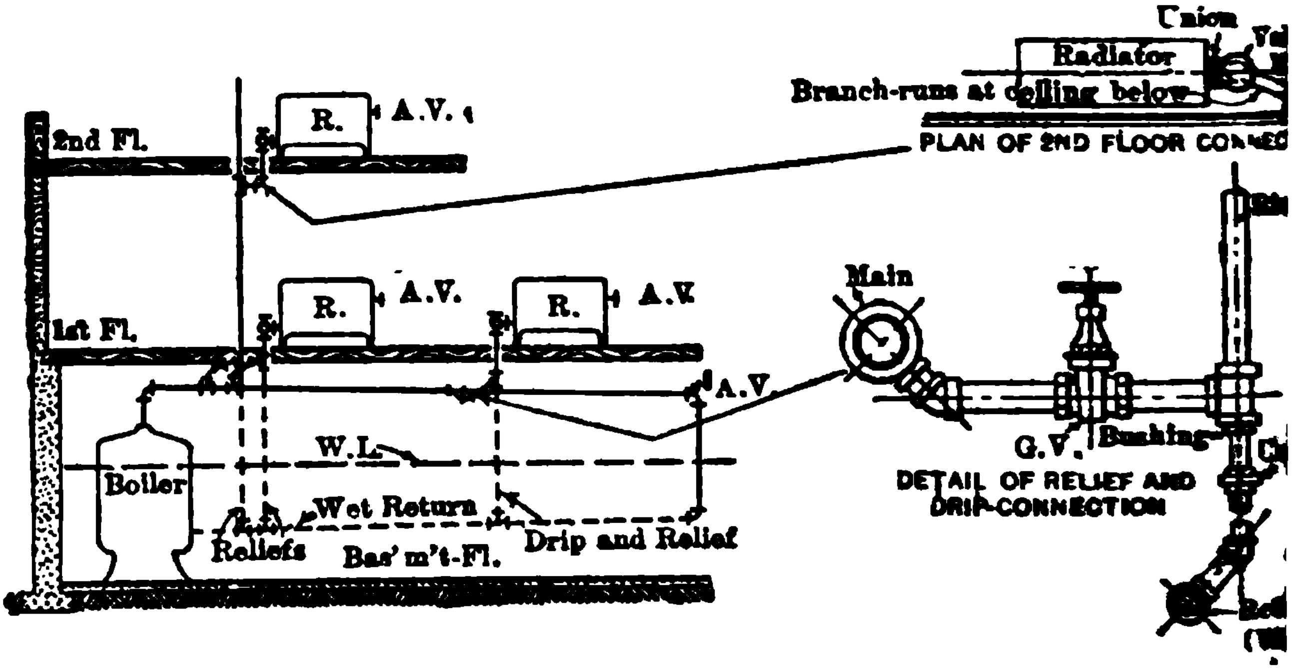


Fig. 21. One-pipe Relief Basement-main

branches. It is customary to maintain at least 18 in between the under sid main at the drop and the normal water-line of the boiler to provide for con gencies. In operation it will be noted that steam and water flow in the s direction through the one-pipe steam-main, and in opposite directions thr the basement-branches, risers, and radiator-branches. This necessitates la piping and valves than in any other steam system, and especially is this t of the main, which must be run full size from boiler to drop, unless dripped shown under piping-details.

ᴜᴍᴘ ꜱʏꜱᴛᴇᴍ (Fig. 21) is very similar to the one-pipe circuit . the risers are dripped individually into the return, and the no radiator-condensation, and is itself dripped at intervals in, which may run ᴅʀʏ or ᴡᴇᴛ. This makes it possible (1) f main as radiation is taken off, (2) to use smaller branches, main much closer to the basement-ceiling, a very important e basement-space is valuable. A ᴄᴏᴍʙɪɴᴀᴛɪᴏɴ ᴏꜰ ᴛʜᴇ ᴏɴᴇ- ᴇ ᴛᴡᴏ-ᴘɪᴘᴇ ꜱʏꜱᴛᴇᴍ is frequently used in large installations, ed for the first and second floors, and the former for the buildings. In this way the amount of condensation flowing risers against the steam is much reduced and smaller risers application of the one-pipe system, with gravity-circulation ts to tall buildings, is not at all unusual, and if the piping is or the circulation of steam and the return of the water of be found satisfactory. In the case of long narrow buildings system it may be necessary to provide a deep boiler-pit so f water in the return-connections will not flood the far end

y Systems. The ᴛᴡᴏ-ᴘɪᴘᴇ ꜱʏꜱᴛᴇᴍ with basement-mains sed in large buildings, and in ᴀʟʟ ᴡᴏʀᴋ ᴡʜᴇʀᴇ ɪɴᴅɪʀᴇᴄᴛ

Fig. 22. Two-pipe Basement-main

ʟᴇᴅ. This system can be readily adapted to mechanical d is very extensively used in this connection. It will be t when applied to a gravity system the return from each ʟʏ ꜱᴇᴀʟᴇᴅ, either by dropping below the water-line to a using drip-loops, as shown at the left, before connecting to in one-pipe work all drips or reliefs are sealed as shown in aution is not taken steam may enter a drip or return from use knocking in the system due to countercurrents of steam ation. Any drip, relief, return-riser, or connection from the ide of the system ᴍᴜꜱᴛ ʙᴇ ꜱᴇᴀʟᴇᴅ. This may be done by water-line, or else by using a running trap or a return-trap onnecting line. Neglect of this precaution will cause an ion of the system.

tor Air-Valves for Gravity Systems. The ᴀᴜᴛᴏᴍᴀᴛɪᴄ ı steam-radiators must be provided for if the highest effi- g-surfaces is to be realized in gravity circulating systems.

Manually controlled air-valves or cocks are usually neglected, and are seldo used for steam-radiators although their use is quite general for hot-water ra ators. Fig. 23 shows a float-type of automatic air-valve. Thermostatic a valves are finding favor in this field. The proper LOCATION OF THE AIR-VAL on a steam-radiator is at the end of the radiator opposite the steam-inlet, and near the bottom of the radiator as possible, since air is heavier than steam the same temperature. In practice, however, the manufacturer of radiato

VALVE CLOSED VALVE OPEN

Fig. 23. Norwall Automatic Air-valve

usually places the air-valve tapping about two-thirds the height of the radiat from the floor in order to prevent possible flooding of the valve.

Special Gravity Systems. In addition to the low-pressure gravity syste already described there are many special steam heating systems known as AI

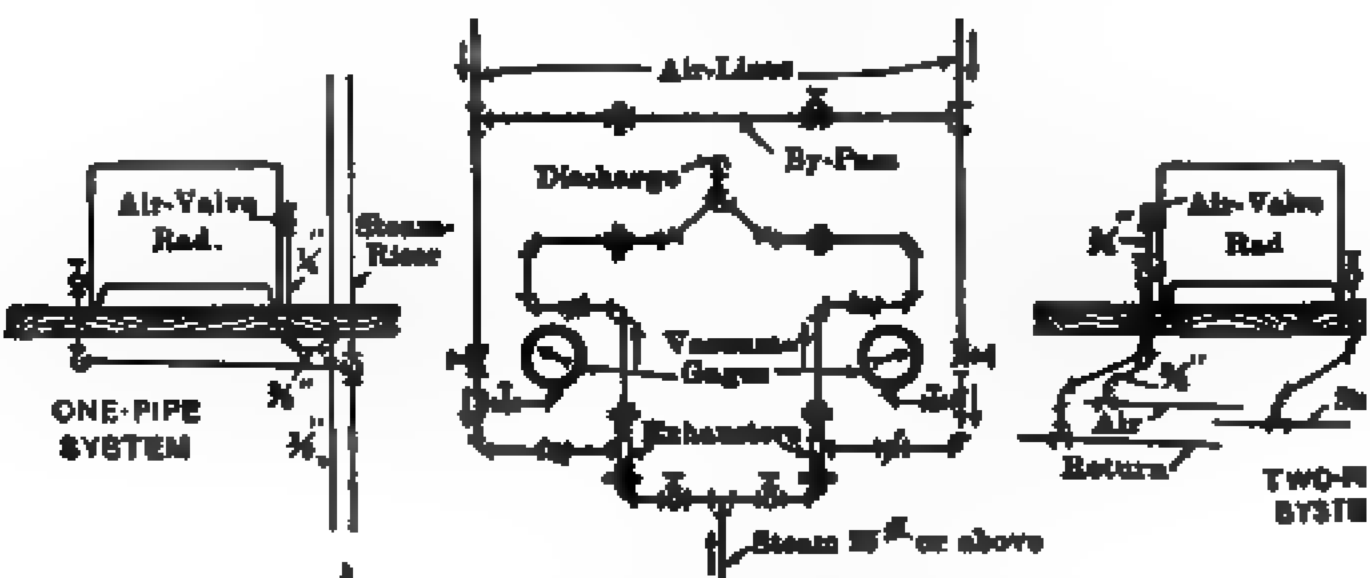


Fig. 24. The Paul Air-line System

LINE, VAPOR, and VACUUM SYSTEMS, also operating with gravity-return of t water of condensation. The AIR-LINE SYSTEM may be attached to any one two-pipe gravity system, and is applied by connecting the automatic air-val of each radiator with small-size piping to an exhauster which maintains a slig vacuum in the air-piping and effectually removes the accumulation of air the radiators. As this scheme is a positive means of air-removal its applicati to the ordinary one or two-pipe gravity system will improve its operati The original air-line system is known as the PAUL SYSTEM (Fig. 24). The e hauster used for less than 2500 sq ft is a water-driven vacuum-pump, with a pr

i at least 20 lb per sq in. Larger systems use a high-pressure steam-jet bove), or if steam is not available, a motor-driven vacuum pump of about se-power, 1 in air-mains in basement, and a gate valve on each air-riser ed. The steam used varies from 1 to 5% of the total condensation. All or-connections are made as shown. The Bishop-Babcock-Becker Com-

Fig. 25. The Bishop-Babcock-Becker Air-line System

anufacture the following line of air-pumps that are used for exhausters ne systems (Fig. 25):

Table XVIII. Hydraulic Exhausters

Diam suction-cylinder, in	Length of stroke, in	Number of pump	City water-pressure, lb	Max. cap. direct rad., sq ft	Number of pump	City water-pressure, lb	Max. cap. direct rad., sq ft
2 ½	4	101	20	700	104	40	4 000
2 ½	4	101	40	800	106	20	6 600
4	[illegible]	102	20	900	106	40	9 600
5 13/16	10	102	40	1 100	2-106	20	14 000
.....	.....	104	20	2 500	2-106	40	20 000

anical Vacuum Systems. The so-called mechanical VACUUM SYSTEMS e two-pipe type, and have a vacuum-pump attached directly to the This pump may be steam or motor-driven, but must be capable of both air and water, as no air-valves can be used on the radiators in the system. The return-end of each radiator is equipped with a radiator-

Table XIX. Motor-Driven Exhausters

Number of pump	Max. capacity, direct radiation, sq ft	Cylinder-sizes		Size of connection		Strokes per minute	Horse-power
		Bore, in	Stroke, in	Disch'e-pipe, in	Suction-pipe, in		
I 279	4 000	2¼	3	1	1	150	¼
112	10 000	3	3½	1	1	70	½
113	18 000	4	3½	1¼	1¼	70	¾
114	28 000	4	5	1½	1½	68	1
115	35 000	5	5	2	2	60	2

trap, usually of the thermostatic type and commonly termed a VACUUM-VALVE, such as the Dunham (Table XX), Webster, Illinois, Monash, etc. A volatile liquid is employed in the thermostatic element or bellows. This liquid is vapor-ized immediately as steam is brought in contact with the bellows, and causes the latter to expand and thus close the valve. The temperature of the con-densate from the radiator is slightly below the temperature of the steam but is not sufficiently high to vaporize the liquid. The valve therefore remains open and will pass the water of condensation and air until the steam starts to flow, when it immediately closes. These valves are very sensitive, and when prop-erly adjusted and in order will not blow steam. One type of THERMOSTATIC VALVE is shown in Fig. 26. It is cus-tomary practice to connect a ½-in cold-water line to the main return at the pump, which serves to condense any steam that may leak by the vacuum-

Connected to

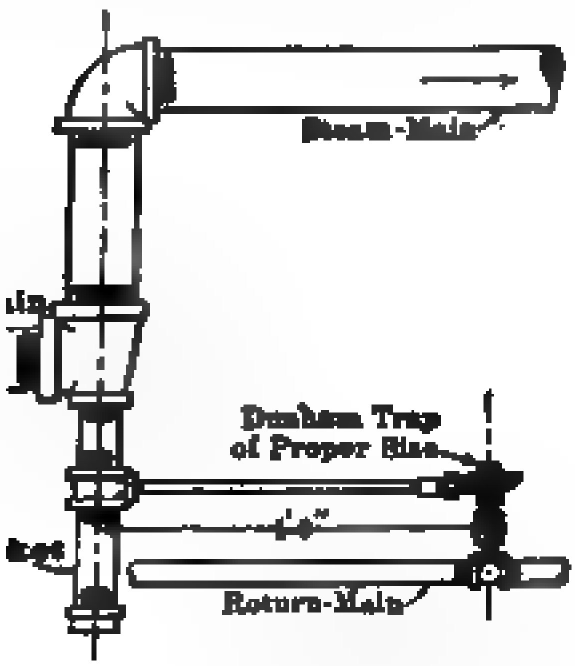
Connected to Return-Pipe

Fig. 26. Thermostatic Valve or Vacuum-trap

Overhead System

valves due to dirt getting under the seat and preventing the valve from closing tight. Figs. 27, 28 and 29 show clearly the application of vacuum-traps to the two-pipe system. It will be observed by inspection of Table XXVII that the return-connections for a vacuum system are much smaller than are used in the ordinary two-pipe system, Table XXVI. The vacuum

ı is largely employed in connection with exhaust steam-heating, where
nportant to keep down the back-pressure on the steam-engines or tur-
to approximately 5 lb per sq in.
ɔass with reducing-valve is used
ss-connect the live-steam main
he heating system. This valve
ıtically opens and allows live
at a reduced pressure (usually
to 5 lb) to flow into the heating



Detail Showing Method of Dripping Rise in Steam-main

Fig. 29. Detail Showing Method of Connecting Rotary Vacuum-pump when Return-line is Below Vacuum-pump

whenever the demand is greater than the supply from the engines, or
e engines are not in operation. (See Fig. 30.)

Radiation

Fig. 30. Exhaust Steam-heating Vacuum System

ıcuum maintained by the pump on the main-return line is ordinarily
in of mercury. This pump is placed under automatic control. The
being operated by the pressure in the return-line

Table XX. Capacities of Dunham Vacuum-Traps

Number	Size, in	Capacity, direct radiation, sq ft	Pipe-connection, in	Weight, lb	Diameter of port, in	Lift, in
1	½	100	½	1 ½	....	....
2	½	350	½	2 ½	⅜	⅕
3	....	450	¾	....	....	....
B.T.	¾	1 500	¾	13	¾	3/16
B.T.	1	3 000	1	21	1	....

These traps are designed for steam-pressures not in excess of 10-lb gauge. For ma and riser-drips, use no smaller trap than the No. 3, and install trap as per details.

Care must be exercised in selecting a trap or traps of the proper size for hot-bla heating-coils. The capacity-ratings for all traps are in terms of direct cast-iron radi tion, on a condensation-basis of approximately 0.25 lb per sq ft per hr. Every unit blast-coil must be reduced to that basis before trap-sizes are chosen and specified. (S Hot-Blast Heating for further details in reference to rating of vacuum-traps for ho blast coils.)

Size of Vacuum-Pump Required. The following table by the Warn Webster Co. may be used in determining the size of steam-driven vacuum-pun necessary. To determine the size of pump required the following empiric formula is used:

Square feet of direct radiation + (number of units × 100) = F. Choo the nearest size corresponding to the value of F given in the table. The stea cylinders are proportioned on a basis of 80-lb pressure and for lower pressur the steam-cylinder must be proportioned accordingly.

Example. Required the size of pump for 5 000 sq ft of direct radiation in 15 radiators. 5-000 + (150 × 100) = 20 000. Use a 5 × 6 × 10-in pump.

Table XXI. Sizes and Capacities of Vacuum-Pumps

Size	Steam, in	Exhaust, in	Suction, in	Discharge, in	F	Floor-space
4 ×5........	⅜	....	....	....	6 830	......
4 ×4 × 6..	⅜	½	2 ½	2	7 270	11×34
4 ×4 × 8..	⅝	½	2 ½	2	8 000	11×34
5 ×5........	....	....	....	....	10 680	......
4 ×5 × 6..	¾	½	3	2 ½	11 353	13×36
4 ×5 × 8..	⅜	½	3	2 ½	12 500	13×38
4½×5½× 8..	½	¾	3	....	15 125	13×38
6 ×5........	....	....	....	....	15 990	......
6 ×7........	....	....	....	....	17 215	......
4½×6 × 8..	½	¾	3	2 ½	18 000	13×38
5 ×6 ×10..	¾	1	4	3	19 390	18×50
7 ×7¼×10..	1	1 ¼	5	5	28 256	18×52
6 ×7½×12..	¾	1	5	4	30 605	19×54
7 ×8 ×16..	1	¼	5	4	34 470	18×52
6 ×8 ×12..	¾	1	5	4	36 620	19×54

This table may also be employed in determining the size of motor-drive reciprocating vacuum-pumps, the last two figures under Size being the diamete

roke, respectively, of the pump. A vacuum-pump should be specified to
ı displacement of at least from 10 to 15 times the volume of the con-
e when operating at its normal rated speed.

Table XXII. Capacity and Size of Steam-Driven Vacuum-Pumps

Water-cylinder diameter, in	Stroke, in	Steam-pipe, in	Exhaust-pipe, in	Suction-pipe, in	Discharge-pipe, in	Draining capacity direct radiation,* sq ft	Draining capacity condensed steam, lb	Floor-space, in
2⅝	4	⅜	½	1¼	1¼	2 700	810	30 × 6
4	6	½	¾	2	1½	7 000	2 100	40 × 10
6	10	1	1¼	4	3	16 000	4 800	59 × 14
10	12	1¼	2	6	5	40 800	12 240	72 × 20
12	12	1¼	2	8	7	62 000	18 600	72 × 20
14	12	1¼	2	10	8	85 000	25 500	....
16	18	1½	2	12	10	92 000	27 600	....
18	18	1¼	2	12	10	128 000	38 400	....

densation figured at 0.3 lb per sq ft radiating-surface per hour.

ım-pumps with belted electric motors are made by the Bishop-Babcock-
Co., with capacities of 2 000, 5 000, 10 000, 17 000 and 25 000 sq ft of direct
ı. This pump should be under the control of a reliable VACUUM-PUMP GOVERNOR
rhen the required vacuum has been produced the pump will stop.

Design of Low-Pressure Steam-Heating Systems

ınt of Radiation Required. The heat-loss, H, of the various rooms is
ed as previously indicated, and H is divided by 250. The result is the
of direct radiation in square feet required. The heat-emission of cast-
iation for pressures up to 5 lb per sq in may be assumed as 250 Btu for
ical purposes of calculation.

g of Boiler Required. If anthracite coal is to be used for fuel add not
ı 50% to the total amount of direct radiation to be installed and 65%
inous coal is to be used, to allow for radiation-loss of boiler, mains,
etc. The steam-mains and risers should always be covered.

of Mains, Branches and Return-Pipes. Steam-mains in low-pressure
systems should be so proportioned that the loss in pressure, due to pipe-
does not exceed approximately 1 oz or 0.062 lb per sq in, per 100 ft of
e reason for thus limiting the pressure-loss is apparent from an inspec-
ig. 31. Owing to the fact that the steam is losing pressure as it flows
the main, it follows that the pressure at the last riser will be lower than
oiler. The difference in pressure, or pressure-loss, P, causes the water
eturn-main to stand higher than the water-line of the boiler. The
ight Z is equal to the height of a column of water which pressure P
ort. Thus, if the boiler-pressure is 2 lb per sq in and the pressure at
ıd of the main is, say 1½ lb, with water weighing 61 lb per cu ft, or 0.035
in, the water in the return will stand $(2 - 1.50) \div 0.035$, or $Z = 14$ in
water-line of the boiler for a ½-lb loss in pressure between the boiler
end of main. It is apparent in this instance that unless the water-line

of the boiler is about 18 in or more below the last riser, or radiator-connecti water is quite likely to flood the steam-main and to be accompanied by a ha mering and a poor circulation in the radiators located at or near the end of t run. Steam-mains are graded in the direction of flow approximately 1 in in 10

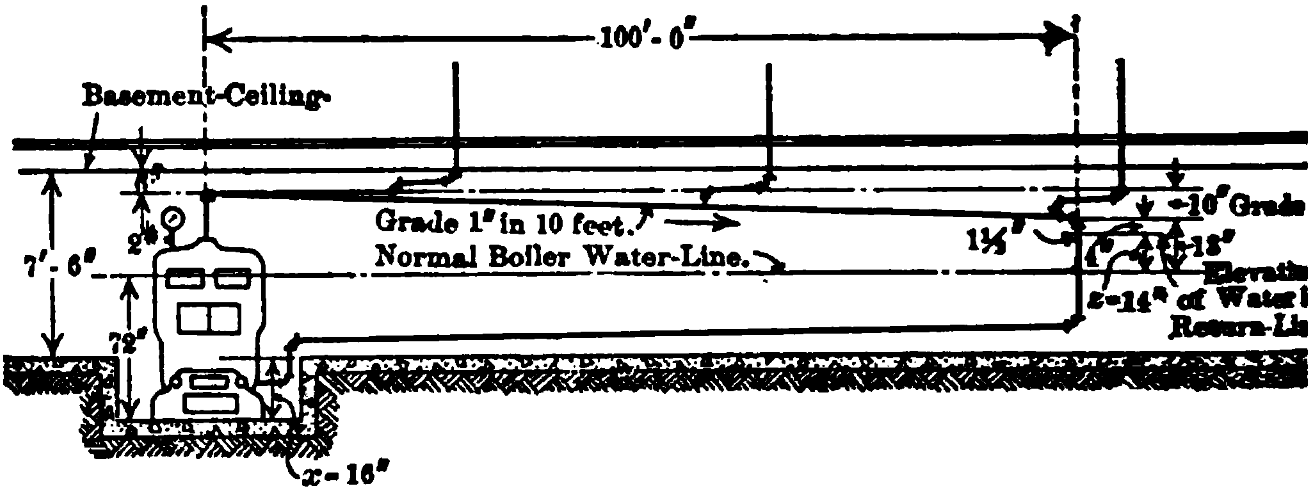


Fig. 31. Location of Boiler and Arrangement of Pipes for Low-pressure Steam-heati Systems

Referring to Fig. 31, and assuming a 7 ft–6 in, or 90 in, clear height of bas ment, and a boiler having a 72-in water-line and a length of steam-main of 100 it is evident that the boiler must be located in a pit, the depth, X, of which

$$6 + 10 + 18 + 72 - 90 = 16 \text{ in}$$

in order to maintain 18 in between the water-line in the boiler and the end steam-main. The distance in practice should not be made less than from to 24 in. The extreme pressure-load stated, ½ lb, in this illustration, is nev approached in normal operation, when the mains are designed for 1-oz dro per 100 ft, but may approach the value stated when the system is being starte up with cold radiators, when the rate of condensation is very much higher. Th pressure-loss in a pipe flowing full of steam may be approximated b Babcock's formula

$$W = 87.5 \sqrt{\frac{ypd^5}{L \times \left(1 + \frac{3.6}{d}\right)}}$$

in which W is the weight of steam flowing per minute in pounds, L the length o pipe in feet, d the diameter of pipe in inches, y the density of the steam and the loss in pressure in pounds per square inch.

One square foot of direct radiation will condense, under normal conditions o operation, 0.25 lb per hr, and the density of steam, y, is 0.043 lb for a 2.3-l pressure. The sizes of steam-mains given in the tables were calculated by th above formula, the pressure-loss, p, being limited to 1 oz, or 0.062 lb per sq i per 100 ft of straight pipe. To allow for the fittings approximately twice thi or ⅛ lb per sq in per 100 ft of pipe may be assumed. The pipe-sizes for the one pipe system are given in Table XXIII, corresponding to the amounts of direc radiation stated in the last column. Branches and risers may be taken fro Table XXIV, and reliefs for risers from Table XXV. For the two-pipe and als the one-pipe relief system the steam-main may be reduced in size as rapidly a the radiation carried will permit. The steam-main should not, however, b of any smaller size than risers called for in Table XXIV.

For one-pipe circuit systems, unless the steam-main is frequently dripped

THE MAIN MUST BE RUN FULL SIZE to the end, at which point an automatic air-valve should be installed and the main dripped into the return. This system is generally used for the heating of residences not exceeding two stories in height. For buildings two stories or more in height the one-pipe relief system

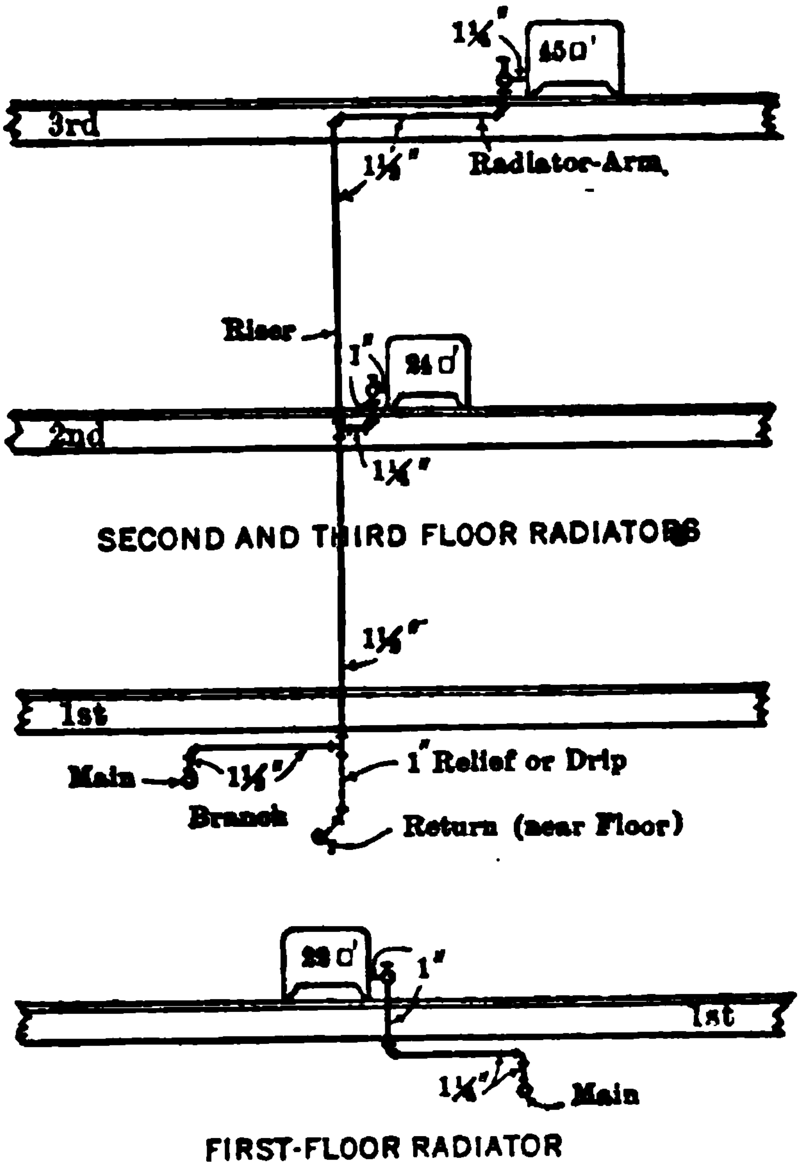


Fig. 32. One-pipe Relief System

Table XXIII. Pipe-Sizes for One-Pipe Low-Pressure Gravity Heating Systems. Main-Table

Steam-main, in	Dry return,* in	Radiation, sq ft
1	1	40
1¼	1	75
1½	1¼	126
2	1½	286
2½	2	535
3	2½	890
3½	2½	1 360
4	3	1 950
5	3	3 600
6	4	5 900
8	4	12 700
10	5	22 900
12	6	37 000

* For wet returns reduce one size, with 1¼ in as a minimum size.

may be employed, in which case the risers supplying the radiation for the seco floor and above are dripped into the return as shown in Fig. 32. The minim size for a wet return should not be less than 1¼ in, as a smaller pipe is likely become plugged with an accumulation of dirt and scale.

Table XXIV. Pipe-Sizes for One-Pipe Low-Pressure Gravity Heating Syste Branch and Riser-Table

Radiation, sq ft	Radiator-tapping, in	Branch-riser, radiator-arm, in
0 to 20	1	1
21 to 24	1	1¼
25 to 40	1¼	1¼
41 to 60	1¼	1½
61 to 80	1½	1½
81 to 100	1½	2
101 to 200	2	2½
201 to 300		3

For risers carrying more radiation than given by the table, use the table f steam-mains and increase one size.

Table XXV. Pipe-Sizes for One-Pipe Low-Pressure Gravity Heating Syste Reliefs for Risers (One-Pipe Relief System)

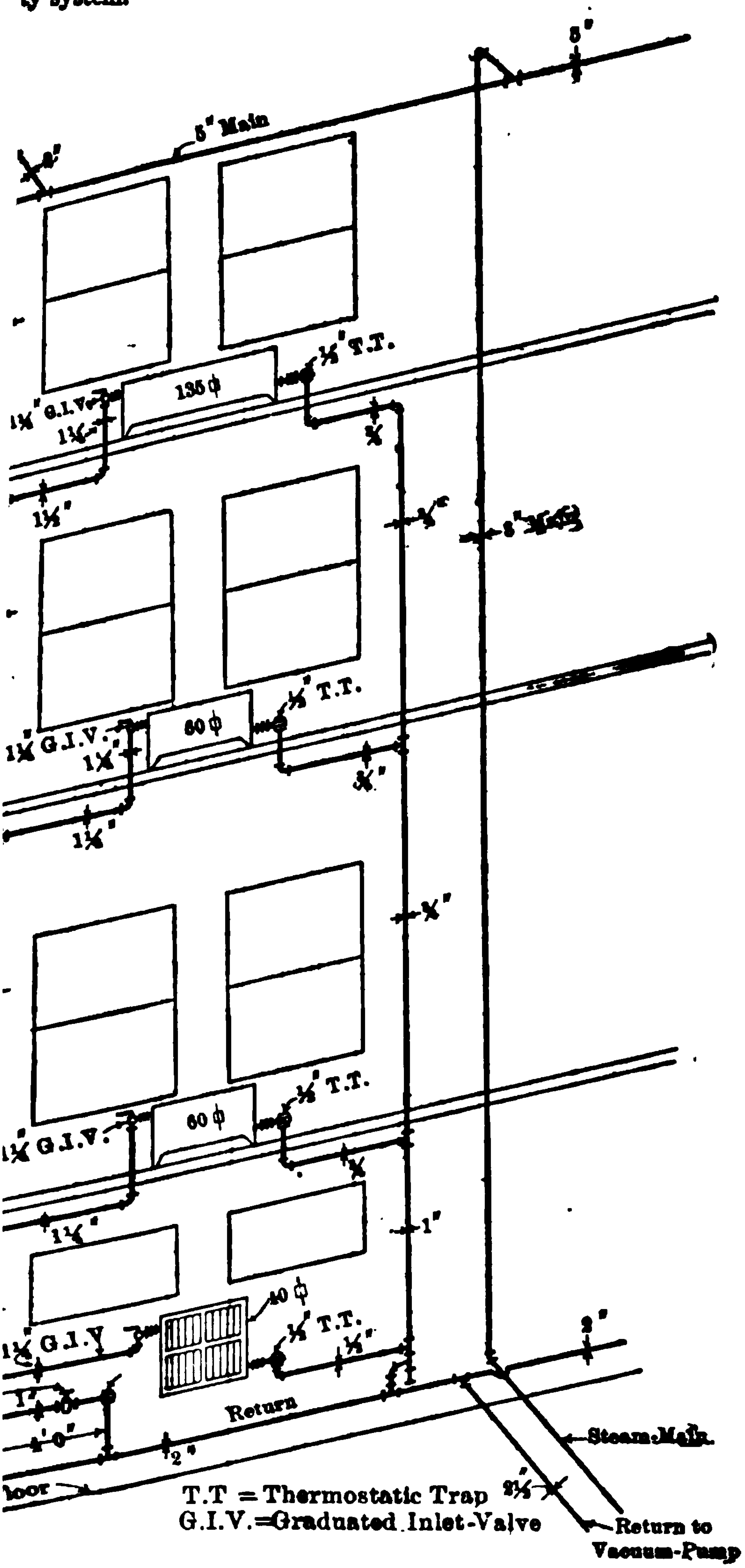
Riser, in......	1	1¼	1½	2	2½	3	3½	4	4½	5
Relief, in.....	¾	1	1	1¼	1½	2	2	2½	3	3

Table XXVI. Pipe-Sizes for Two-Pipe Gravity Systems

Diameter of supply, in	Diameter of dry return,* in	Direct radiation-surface supplied, sq ft
¾	¾	20
1	¾	36
1¼	1	72
1½	1¼	120
2	1½	280
2½	2	530
3	2½	900
3½	2½	1 320
4	3	1 920
4½	3	2 760
5	3½	3 720
6	3½	6 000
8	4	12 800
9	4½	17 800
10	5	23 200
12	6	31 000

* For wet returns, reduce one pipe-size, with 1¼ in as a minimum.

Two-Pipe Low-Pressure Gravity Heating Systems.
e used in determining the sizes of mains, branches and risers
ty system.



r-diagram for Down-feed Mechanical Vacuum System

Pipe-Sizes for Two-Pipe Mechanical Vacuum Systems. The si[illegible] of branch-supplies to radiators, risers, steam-mains, radiator-returns with th[illegible] mostatic valves and main-returns may be taken from Table XXVII. (S[illegible] Example, Fig. 33.)

Table XXVII. Two-Pipe Mechanical Vacuum Systems

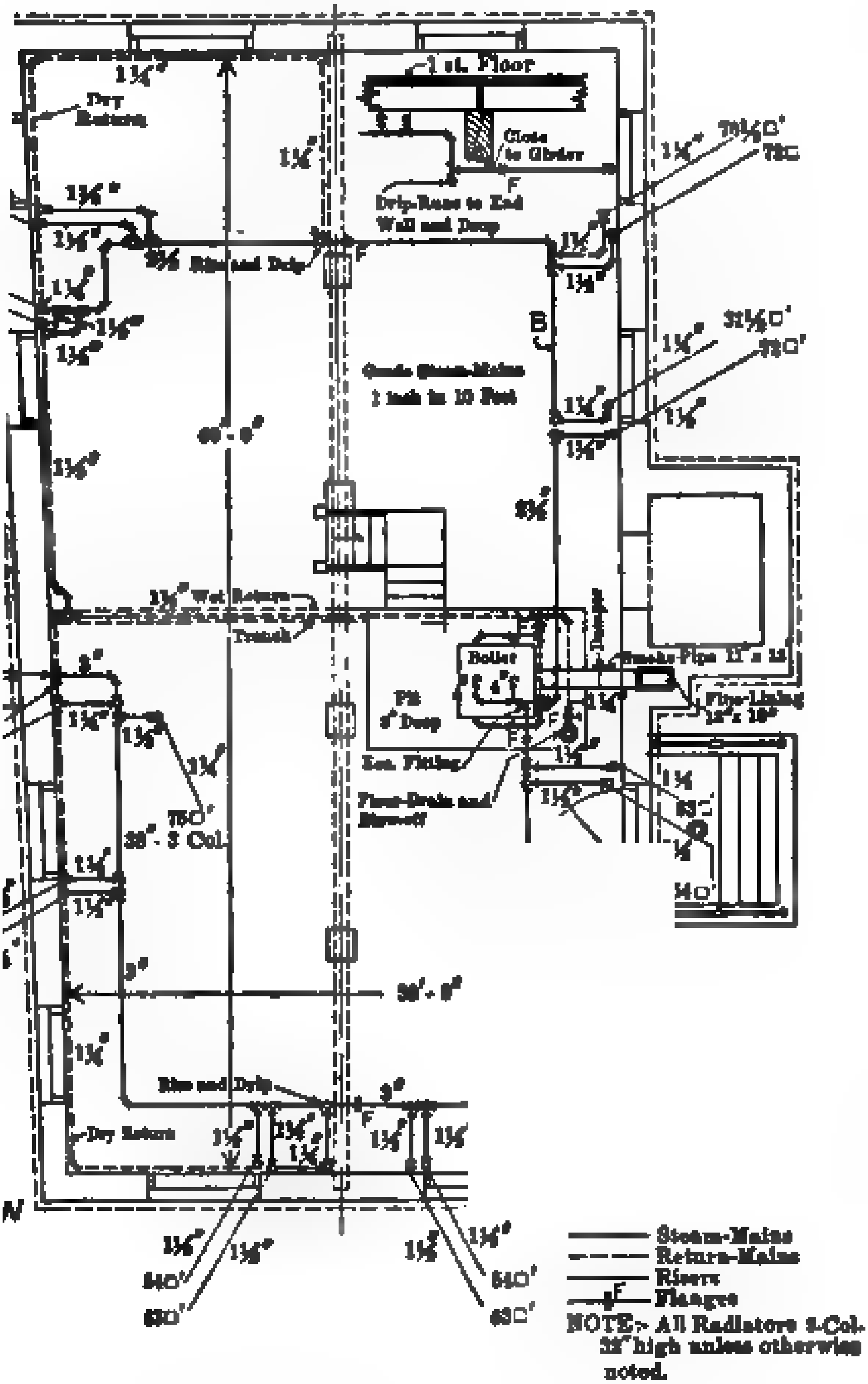
Size of pipe, in	Rating direct radiation: Steam-mains and risers, sq ft	Rating direct radiation: Return-mains and return-risers, sq ft	Radiator-connections: Size of radiator, sq ft	Radiator-connections: Size of steam-connection, in	Radiator-connections: Size of return-connectio[n] and valve in
½	5	40	30	¾	½
			50	1	½
¾	20	160	75	1¼	½
			100	1¼	....
1	40	320	125	1½	½
			150	1½	....
1¼	75	600	200	2	½
1½	150	1 200	300	2	¾
2	300	2 400	....	....	....
2½	500	4 000	....	....	....
3	900	7 200	....	....	....
3½	1 500	12 000	....	....	....
4	2 000	16 000	....	....	....
4½	2 800	22 400	....	....	....
5	3 600	28 800	....	....	....
6	6 000	48 000	....	....	....
8	13 000	72 000	....	....	....
9	18 000	....	....	....	....
10	23 000	....	....	....	....
12	37 000	....	....	....	....
14	55 000	....	....	....	....

Table XXVIII. Direct Radiation Required for the Factory Office-Building Sho[wn] in Figs. 9 and 34.

Room-designation	Btu-loss per hr	Direct rad'n required, column 2 ÷ 250	Radiation to be installed: No. radiators	Radiation to be installed: No. cols. and height	Radiation to be installed: No. sect's each radiator	Radiation to be installed: Total, sq ft
1	2	3	4	5	6	7
Sample rm.......	54 900	219	4	3–32″	12	216
Hall............	19 123	77	1	3–38″	15	75
Laboratory......	27 358	95	2	3–32″	11	99
Storage.........	19 482	77	1	3–32″	17	76
Toilet..........	7 380	29	1	3–32″	7	31
Mgr's office......	31 306	125	2	3–32″	14	126
Hall............	19 224	78	1	3–38″	16	80
Gen'l office	70 247	281	4	3–32″	16	288
Sup't's office	31 306	125	2	3–32″	14	126
Totals.......	......	......	......	......	......	1 108

'or Indirect Gravity Radiation. Multiply square feet of a by 2 and use Table XXVI.

)ne-Pipe Low-Pressure Gravity Heating System. Direct t it be required to design a heating system of this type



ıngle-pipe Low-pressure Steam-heating System for Factory Office-building. (See, also, Fig. 9.)

tory office-building shown in Figs. 9 and 34, the heat-losses being as calculated and given by Table VII. The location of the radiators cated on the floor-plans, Fig. 9.

Direct Radiation Required. The total Btu-loss for each room is repeat in column 2 of Table XXVIII. The amounts of radiation required for these los is given in column 3 and were obtained by dividing the Btu-loss in each case b 250, the average heat-emission of direct radiation (Btu) per square foot p hour for a room-temperature of 70° and 2 lb steam-pressure.

Boiler-Capacity Required. The boiler-capacity must be such that it wi carry the radiation installed plus the extra heat-loss from the steam-mai return-mains and boiler. The steam-mains are to be covered and anthraci fuel is to be used. The fuel-capacity of the fire-pot is to be sufficient for an hr firing-period. Adding 50% to the sq ft of radiation installed, 1.5×1.1 = 1 662 sq ft, the boiler-rating required. The fuel-holding capacity of th fire-pot should be checked for the boiler proposed as previously stated und Boilers.

Chimney-Size. The size of chimney may be taken from Table XVII, an in this case, is 12×12-in inside dimensions, by a 40-ft height. The neare size for flue-lining, Table XVI, is 13×18 in.

Design of Piping System. The layout of the basement-mains and rise for a ONE-PIPE CIRCUIT SYSTEM is given in Fig. 34. The risers are not drippe in this case. Steam-main *A* supplies 623 sq ft of radiation, and, according t Table XXIII must be 3 in in diameter. The diameter must be carried to th end where it is dipped into the return-main by a 1½-in relief, as indicated i Table XXIII, for a dry return-pipe. In order to keep the steam-main as clo to basement-ceiling as possible, where it passes beneath the floor-girder, a ri is made, and consequently a 1¼-in drip must be provided to take care of 2 sq ft of radiation. The main wet return takes care of 1108 sq ft of radiatio and is therefore made 2-in diameter. The risers and branches are proportione by Table XXIV.

Gravity Indirect Heating

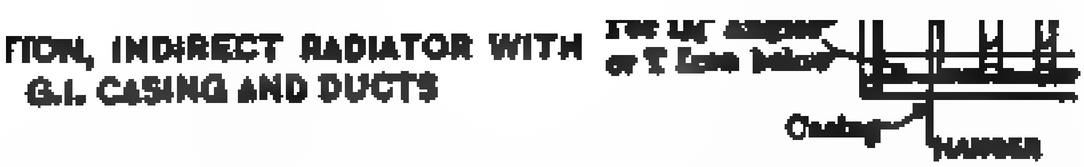
General Description. A satisfactory means of providing for the heat-lo in a room, and, at the same time, supplying air-ventilation, is accomplished b this system. The radiators properly encased with a sheet-metal casing, cov

Table XXIX. Final Temperature of Air Passing Over Indirect Radiation. Ex tended-Surface Type. Initial Temperature of Air, 0° F. Heater, On Stack in Depth. Four-Inch Spacing of Sections

Velocity of air through free area of heater, in ft per min, v*	Final temperature, t_2, in degrees Fahrenheit	
	Steam at 2-lb pressure	Hot water, 180° F.
50	122	147
100	100	127
125	95	120
150	90	113
175	86	106
200	82	102

* Measured at 70° F. For first-floor registers a velocity of 150 ft per min throug ree area and 150 ft per min for second-floor registers is the usual assumption.

on, are ordinarily hung from the basement-ceiling by means of r strap-iron, as shown in Fig. 35. Each radiator is ordinarily



35. Indirect Radiators, Casings, Connections, etc.

sh-air inlet and hot-air duct connecting the radiator with the ecirculating duct may be provided, as indicated, in order to heating in ex- er if desired. separate ver- each register nnected with diator. At- y more than an indirect ally success- ed, unless a yed to give as with the described ducts for best results, pe as later ce-heating. is designed

Fig. 36. School Pin Indirect Radiator for Steam or Water

num of heating-surface to the air passing over same. standard types for gravity indirect heating may be men- Pin Radiator (Fig. 36), Excelsior, Sterling and Vento. e now rated according to the temperature-increment, or

rise, which they are capable of giving to the air passing between and over th sections of the heater for various velocities of air, initial temperature, and tem perature or pressure of the steam, or temperature of the hot water The velo ities stated are, for convenience in rating, based on air at 70°. The free or unob structed area means the net area between heater-sections after deducting th area of the projecting surfaces from the gross area. Limitations of space pr vent giving more than these data for one type of indirect radiator.

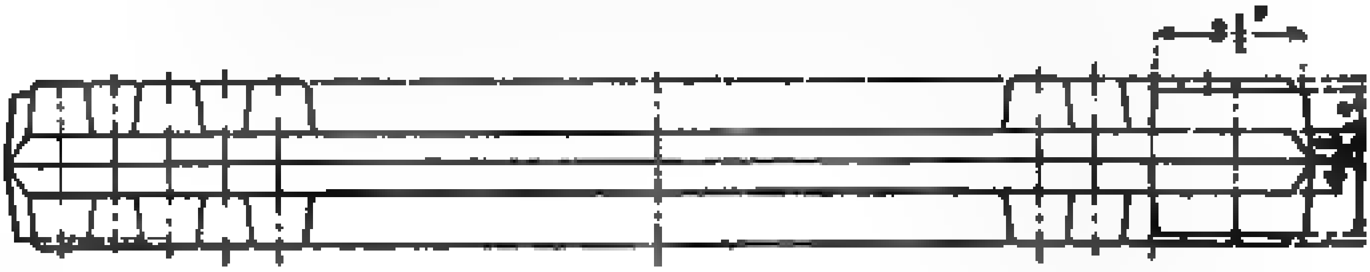


Fig. 37. Vento Thirty-inch Indirect Heater-section

Tables XXIX and XXX give the results of tests made on Vento, indire radiation, American Radiator Company. (See Fig. 37.)

Table XXX. Dimensions, etc., Vento Indirect Radiation

Size	Heating-surface, s, sq ft	Height, in	Width, in	Free area between sections, (a) sq ft
30-in section....	8.00	$29\frac{7}{8}$	$9\frac{1}{4}$	0.256
40-in section ...	10.75	$41\frac{1}{64}$	$9\frac{1}{8}$	0.35
50-in section......	13.50	$50\frac{29}{32}$	$9\frac{1}{8}$	0.423
60-in section ...	16.00	$60\frac{15}{16}$	$9\frac{1}{8}$	0.511

Spacing of sections, 4 in on centers for gravity air-circulation.

Weight of Air to be Circulated per Minute. W. t_1 = temperature of leaving register. t = temperature of room. 0.24 = sp heat of air. H = he loss of room to be warmed in Btu per hour. V = volume of air in cubic f per minute measured at 70°.

Then $$W = H/[60 \times 0.24(t_1 - t)], \text{ lb of air per min}$$
$$V = W/(0.075 = \text{the density of the air at } 70°)$$

ea of Indirect Heating-Surface Required, S. t_0 = initial temperature of ıtering indirect heater. t_2 = final temperature air leaving heater = $t_1 + 5°$ med temperature-loss in hot-air duct). V = velocity through free area of r in feet per minute, measured at a temperature of 70°. F = total fire-equired in heater, in square feet. a = free area for one section of heater ıare feet. n = number of sections required.

$$F = V/v = W/0.75v, \; n = F/a \text{ and } S = n \times s$$

mple. Required the amount of indirect low-pressure steam-surface of ıtended-surface type, the number and size of sections, and the over-all sions of an indirect radiator to supply the necessary heat to warm a oor room, the heat-loss of which is H = 20 000 Btu per hour. All the air e taken from the outside, the temperature of which is $t_0 = 0°$ F. The temperature to be maintained is $t = 70°$ F.

tion. It is first necessary to assume a temperature, t_1, for the air entering om, in order to calculate the amount or weight, W, of air required to be ted to convey the heat required to make up the heat-loss H.
ıme $t_1 = 95°$ and $t_2 = t_1 + 5$ (loss) = 100°; and v = 100 ft per min. 'able XXIX.

$$W = 20\,000/[60 \times 0.24(100 - 70)] = 46.3 \text{ lb per min}$$
$$V = 46.3/0.75 = 617 \text{ cu ft per min, measured at } 70° \text{ F.}$$

me 40 in as the length of section desired in this installation,

a = 0.35 (Table XXX);
F = 617/100 = 6.17, the total square feet of free area required;
n = 617/0.35 = 18, the number of sections of 40 in.

o is, therefore, required, giving a total heating-surface of

$$S = 10.75 \times 18 = 193.5 \text{ sq ft}$$

g this equally between two indirect radiators the width of each heater to nine (sections) × 4 (spacing-sections) = 36 in.

Pressure Boiler-Rating Required for Gravity Indirect Radiation. ount of heat given up to the radiator is

$$h = 0.24W(t_2 - t_0) \times 60 \text{ Btu per hr}$$

equivalent rating in square feet of direct radiation is therefore R = lus 25% for radiation of mains, returns, etc.

ple. Required the equivalent low-pressure boiler-rating to supply the radiation in preceding example.

on.

$$h = 0.24 \times 46.3\,(100 - 0) \times 60 = 66\,672 \text{ Btu per hr}$$
$$R = (66\,672/250) \times 1.5 = 399 \text{ sq ft}$$

ıer words, 1 sq ft of low-pressure steam indirect radiator with gravity-on is practically equivalent to 2 sq ft of direct radiation; or the amount ct surface is approximately 0.4 of the amount of direct radiation

of Hot-Air Ducts for Gravity-Circulation. A velocity of approxi-ne third of the theoretical velocity attainable by natural draft, due to

the smaller density of the heated air, is assumed in practice in proporti the area of the hot-air ducts.

Table XXXI. Theoretical Velocity (V) of Air, in Feet per Second, Due to Natu Draft

Height of flue in feet, E	Excess of temperature in flue above external air							
	10°	15°	20°	25°	30°	50°	100°	150°
1	1.1	1.4	1.6	1.8	2.0	2.5	3.6	4.4
5	2.5	3.1	3.6	4.0	4.5	5.6	8.1	9.9
10	3.6	4.4	5.1	5.7	6.6	8.1	11.4	14.0
15	4.4	5.4	6.3	7.0	7.7	9.9	14.0	17.1
20	5.1	6.3	7.2	8.1	8.8	11.4	16.1	19.8
25	5.7	7.1	8.1	9.0	9.9	12.8	18.0	22.1
30	6.3	7.8	8.8	9.9	10.8	14.0	10.8	24.2
35	6.8	8.4	9.5	10.7	11.7	15.1	22.3	26.1
40	7.3	8.9	10.2	11.4	12.5	16.1	22.8	27.9

Example. Required the size of hot-air duct for each of the indirect radiat in the preceding examples for a first-floor installation, the effective height E bei 5 ft.

Solution. The excess of temperature in the flue above the external air 100 − 0 = 100°. The theoretical velocity in the duct, from Table XXXI, 8.1 × 60 = 486 ft per min. The actual velocity is approximately one third this, or 162 ft per min. The weight of air per minute passing through the flue 23 lb, or

$$23/[0.071(\text{density at } 100°)] = 324 \text{ cu ft}$$

The required area of the flue is therefore

$$324/162 = 2 \text{ sq ft} = 288 \text{ sq in}$$

The gross area of the register-face must be approximately 1.8 this amou or 518 sq in, to obtain the same free area through the register-grill as exists the flue or duct. Sizes of standard registers are given in the section on Fu nace-Heating.

Direct Hot-Water Heating

Systems in Use. Systems for heating with direct hot-water radiators, l the direct steam heating systems, may be divided into two general classes, t first of which includes all those systems OPERATING BY GRAVITY ONLY, dependi on the difference in density of the water-columns in the flow and return-lines cause circulation. The second class includes those systems in which a FORCE CIRCULATION is maintained by means of a pump placed on the return-line ju before it enters the boiler or heater. These latter systems are employed usual only in large installations or in district-heating service.

Gravity Hot-Water Heating Systems. The gravity systems are divid into the UPFEED SYSTEMS, using basement-mains, and the DOWNFEED SYSTE using overhead or attic-mains. The upfeed systems may have either a one-pi basement-main or two-pipe basement-mains, and the latter type may ha either a DIRECT or a REVERSED return-main. (See Figs 38 and 39 for revers

ifeed systems may have either SINGLE OR DOUBLE RISERS.
be operated with an OPEN or CLOSED EXPANSION-TANK,
? to 46. In general, the downfeed or overhead systems are
it the use of smaller mains and risers, and provide for the
of air from the radiators and piping. It is necessary, how-
oom or clear space in the attic should be at least 4 or 5 ft if
and branches are to be properly installed. It is sometimes
verhead mains at the ceiling of the top floor, and in such
striction does not apply. Mains run in attics must be
revent freezing. The underfeed systems are used where

BASEMENT HEATING PLAN

d 41. Hot-water Heating. Two-pipe Up-feed System

ilable, and of little or no value, and the radiation is
floors; or where attic-space is so limited that it would
all overhead mains and branches. Underfeed systems
tisfactory in buildings less than two stories in height, as
adiators on the first floor only is so slight that faulty or
quite likely to result.

ipe System. The upfeed one-pipe system is in very
is employed almost exclusively by the United States
artments whenever upfeed hot-water systems are to be
m, as shown in Figs. 43 and 45, the supply-main rises
iling just above the boiler and grades down in the direc-
orm grade of ¼ in in 10 ft. Branches are taken from the
plying flow-risers and the return-branches are made into

the side or bottom. (Fig. 40.) Flow-connections should always be made fr the top, or at an angle of 45° in the case of branches near the boiler, or for branc

HOT-WATER HEATING - EXPANSION TANKS
(OPEN AND CLOSED SYSTEMS)

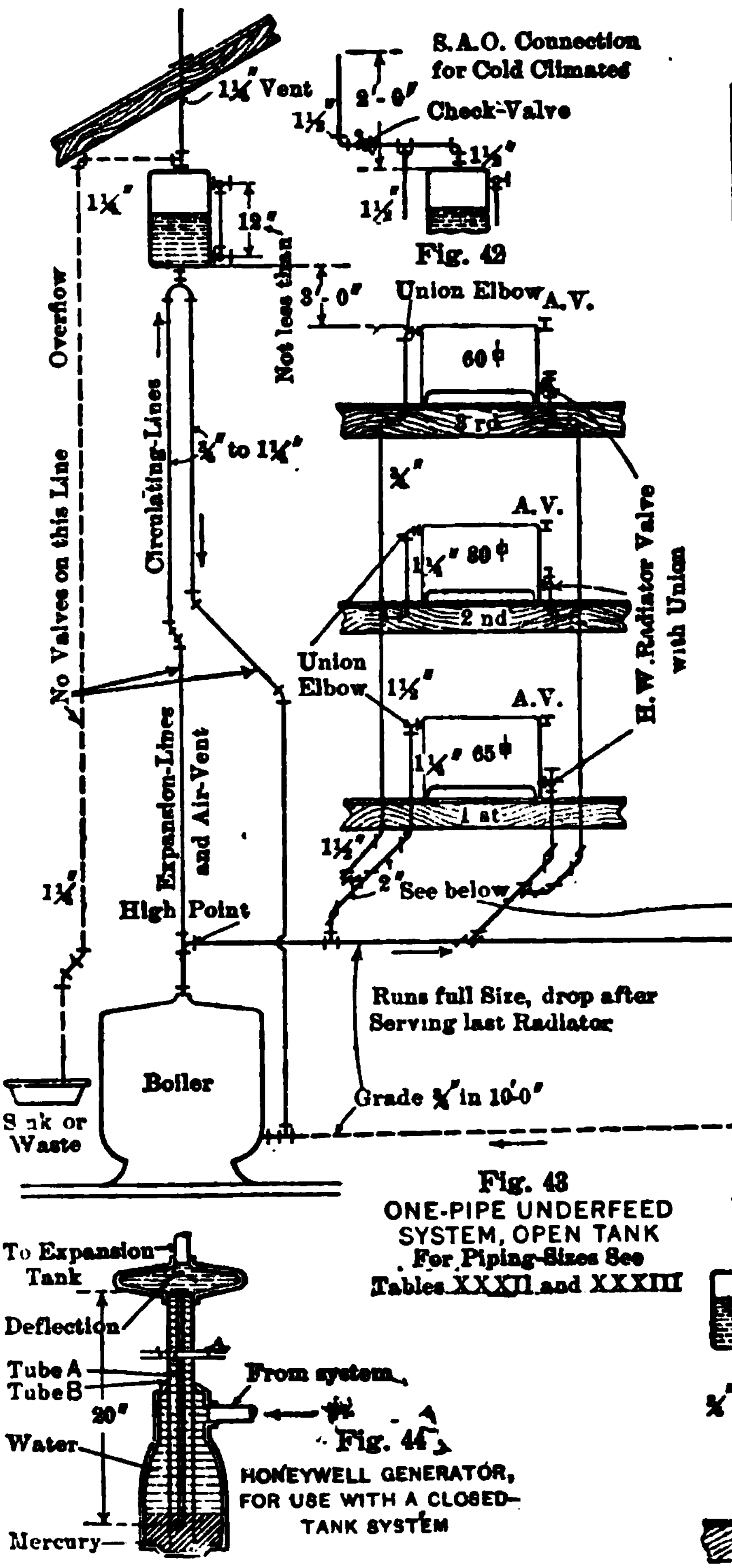


Fig. 42

Fig. 43
ONE-PIPE UNDERFEED SYSTEM, OPEN TANK
For Piping-Sizes See Tables XXXII and XXXIII

Fig. 44
HONEYWELL GENERATOR, FOR USE WITH A CLOSED-TANK SYSTEM

Table ...
Expansion-Tanks

No.	Size in Inches	Capacity Gal.	Sq. ft. Rad-iation
0	10 x 20	8	[illegible]
1	12 x 20	10	[illegible]
2	12 x 30	15	[illegible]
3	14 x 30	20	[illegible]
4	16 x 30	26	[illegible]
5	16 x 36	32	[illegible]
6	16 x 48	42	[illegible]
7	18 x 60	66	[illegible]
8	20 x 60	82	[illegible]
9	22 x 60	100	[illegible]

Note.- Galvanized Steel, Teste at 100 lb. Tapped 1¼" Top and Bottom, and for Gauge-Glass

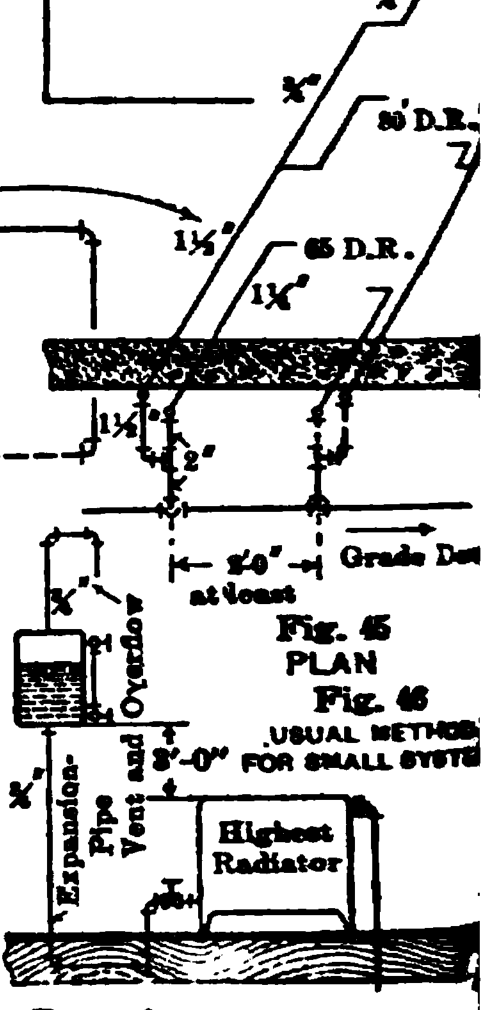


Fig. 45
PLAN

Fig. 46
USUAL METHOD FOR SMALL SYSTE

Figs. 42, 43, 44, 45 and 46. Hot-water Heating. Expansion-tanks. One-pipe Underfeed System

supplying only upper-floor radiators. It will be seen that in the case of branc supplying radiators on all floors the upper-floor radiators may be made to r

culation of the first-floor radiators by taking the basement-ter from the side of the branch running to the latter radiator. ach is usually run full size all the way to favor the lowest in Fig. 45. After having served all the radiator-branches nd returns to the boiler, continuing the same size for the nnections (Fig. 43) to radiators should be made at the TOP using a union elbow, and at the BOTTOM on the return-end, ing hot-water radiator-valve with union connection. By nly one valve is required to control the radiator. Since the water in the one-pipe main gradually drops, due to the return temperature from the radiators served in the course of the uilding, it is advisable to increase the last radiators on the o in area and to increase the size of branch and riser-connec-the main by a one-pipe size. Pipe-sizes may be taken from XXXIII. In using the tables all mains must be measured and risers to any floor are proportioned to supply all the it floor as well as the radiator actually installed on that Figs. 43 and 45.

-Water Heating. Piping-Sizes. Open-Tank (Upfeed with Basement-Mains

Mains up to 100 ft

Pipe-size in inches	Direct radiation in square feet	Indirect radiation in square feet
1¼	135	100
1½	220	135
2	350	225
2½	460	320
3	675	500
3½	850	650
4	1 100	850
4½	1 350	1 050
5	1 700	1 350
6	3 600	2 900
7	4 800	3 900
8	6 200	5 000
9	7 700	6 300
10	9 800	7 900
12	14 000	11 400

ompiled by J. J. Hogan, and is to be used for either one or two-

th of main must be measured back to boiler.

over 100 ft, reduce capacity in the ratio of $\sqrt{\frac{100}{L}}$.

-Pipe System. The upfeed two-pipe system is also in if installed with a REVERSED RETURN, as shown in Figs. od results. If a DIRECT RETURN is used so that the water the radiators nearest the boiler, and then through each rn, the ends of mains will be slow in warming up, the last , and the system prove unsatisfactory. With the RE-

Table XXXIII. Hot-Water Heating. Piping-Sizes. Open-Tank (Upfeed w Basement-Mains)

Branches and Risers

Pipe-size in inches	Floor			
	First	Second	Third	Fourth
	Direct radiation in square feet			
¾	30	45	55	70
1	60	75	85	95
1¼	110	120	135	150
1½	180	195	210	230
2	290	320	350	370
2½	400	490	525	550
3	620	650	690	730
3½	820	870	920	970
4	1 050	1 120	1 185	1 250
4½	1 325	1 400	1 485	1 560

Table XXXIII was compiled by J. J. Hogan, and is to be used for either one or t pipe work.

VERSED-RETURN SYSTEM each group of radiators has exactly the same lengt water-travel, and hence the resistance to be overcome is practically the sa irrespective of the distance of the radiator-group from the boiler. It wil noted that the return begins at the first radiator served and flows in the s DIRECTION as the flow-main, increasing in size while the latter decreases. flow-main grades UP uniformly ¼ in in 10 ft, and the return grades DOWN to the boiler with the same pitch. Pipe-sizes may be taken from Tables XX and XXXIII as in one-pipe work, and the main-size reduced or increased rapidly as the change in radiation supplied will permit. It is also custo in government work to install a STARTING-PIPE (Fig. 38), between the main and return at the boiler, in underfeed systems. This pipe ranges from 1¼ 2½ in in size, depending upon the capacity of the boiler, and is intended to a in the establishment of an initial circulation between flow and return-head even before the water in the mains is moving.

Equalization-Table. In Federal-building work N. S. Thompson makes of the following Equalization-Table in proportioning mains and risers serv more than one radiator in both upfeed and downfeed systems. The eq izing-numbers represent the relative capacities of the different sizes of pipes

Table XXXIV. Equalization-Table for Mains and Risers

in	in	in	in	in
½ = 2	¾ = 5	1 = 10	1¼ = 20	1½ = 3
2 = 60	2½ = 110	3 = 175	3½ = 260	4 = 3
5 = 650	6 = 1 050	7 = 1 600	8 = 2 250	....

Example. A 1¼-in, 1½-in and 2-in pipe have a total value of 110 units, and are equivalent in carrying capacity to a 2½-in main.

me friction-pressure loss per 100 ft of run, and are proportional to the 5/2 s of the diameters. Thus the weight of water flowing varies as shown by lation, $W = Kd^{5/2}$, in which W = weight, I = a constant, and d = pipe-ter.

ails of Piping Systems and Connections for Direct Hot-Water ng. The distinctive piping-details of each system of hot-water heating been discussed under that system, as described in the preceding para-. In general all main piping and branches must be UNIFORMLY GRADED, ady indicated, and ample provision made for EXPANSION and CONTRACTION, ready REMOVAL OF AIR from all parts of the system. Air-traps or pockets t-water system are fully as serious as water-pockets in a steam system. a hot-water main grading down in the direction of flow cannot be relayed an air-outlet is provided at the top of the relay. If the main is reduced at any point an ECCENTRIC FITTING must be used to keep the TOP of the nd small main in the same plane and avoid an air-pocket. Not only l the piping be designed to permit the removal of air, but FREE AND COM-DRAINAGE of water must be provided for as well, so that when the drain -off cock is opened at the boiler the entire system can be emptied of water. ch-mains are taken from a HEADER at the boiler they must all rise to the LEVATION so that the tops of all the branches will lie in the same plane start away from the boiler. The fittings on all main piping and branches of the long-sweep pattern, and all pipe should be carefully reamed to burrs and sharp edges. Where the same riser supplies radiators on two floors the branches to the radiators on the intermediate floors may be ed with special tees (Fig. 41) known as O. S. FITTINGS, with a deflector l to divert the current of flow into the outlet of the tee, and thus favor iators on the intermediate or lower floors. By using TOP-FLOW and RETURN CONNECTIONS at each radiator it is possible to positively control t by a single valve, except for the slight circulation intended to prevent which takes place through the 1/16-in-diameter hole drilled in the valve-leeve, when the valve is closed. If both connections are made at the tappings, and only one valve is used, it is entirely possible that the may still be supplied with hot water through the unvalved connection en the valve is closed.

emoval in Hot-Water Systems. Suitable provision must be made for val of air from all hot-water radiators, wherever an upfeed system is Usually small air-cocks are attached to the highest point of each and are periodically opened to relieve any accumulation of air. If ks are forgotten a radiator may become air-bound and fail to heat f faulty circulation; hence automatic air-valves are sometimes installed purpose. The automatic air-valve for hot-water radiators is not very used, due to its liability to pass water as well as air; but a standard le by the Monash-Younker Company, may be mentioned.

sion-Tanks for Hot-Water Heating. Open-Tank Systems. The are system of hot-water heating is not a closed system, as provision made for expansion and contraction of the water within the system. ank is provided at a suitable elevation, not less than 3 ft above the diator, and connection made to the nearest return-riser; or preferably expansion-line is run to the flow or return-main in the basement. F THE EXPANSION-TANK varies with the amount of water in the system, vith the range in temperature of same, and its capacity is determined

rease in volume of a given weight of water heated from 32° to 212° is

about $\frac{1}{23}$, or approximately 4.33%; so that for every 23 gal in the system 32°, an allowance of 1 gal must be made in the expansion-tank when the wa in the system is raised to 212°. Cast-iron radiators have an internal volume 1½ pints per sq ft, while steel radiators and 1-in pipe hold about 1 pint per sq Assuming the internal volume of the radiators to be about 50% of the en system, we have for 3 000 sq ft of actual radiation, 3000 × 2 × ⅛ gal = gal of water. This water will increase $\frac{1}{23}$ × 750 = 33 gal on being hea from 32° to 212°. Hence an expansion-tank of 2 × 33 = 66-gal capacity necessary, the tank being made double the theoretical volume for practi considerations.

A list of expansion-tank capacities and dimensions is given in a ta (included with Figs. 42 to 46), from which a commercial tank may be read selected for systems under 6 000 sq ft. For larger systems the size of ta should be separately determined and the nearest commercial tank-size, as tal from the manufacturer's list, should be specified. These tanks should hav or 1¼-in top and bottom tappings with ½-in water-gauge tappings, for c necting a gauge-glass, at least 12-in long, on the side of the tank as shown Fig. 43. The tank must be securely supported well above the highest radia in the system, and in the larger installations special framing must often designed to carry the weight of tank and water. Automatic expansion-ta equipped with a ball-cock and overflow are sometimes installed, and the altitu gauge on boiler, and the gauge-glass and fittings on tank omitted. These ta may be covered with hardwood and varnished if it is necessary to place them i finished room or apartment.

Expansion-Tank Connections. The most approved method of connect an expansion-tank to a low-pressure one-pipe system is shown in Fig. 43, wh an expansion-and-vent line is run from the top of the main, at the high p just above the boiler, and connected to a return-bend just beneath the ta A return circulating-line is taken from the other side of this bend and connec with the return-main at the boiler. The circulation of water in this loop prevent freezing at the tank. From the top of the tank a 1¼-in vent-lin taken through the roof, and a 1¼-in overflow is taken out of this vent-line tee just above the tank. This overflow should discharge into an open sink drain near the boiler so that it will be immediately evident to a person in boiler-room, filling the system, just when the water has risen to the overf above the tank. The movable hand on the boiler altitude-gauge can then be to correspond with the middle of the gauge-glass, and the water-level brou to this point with the system cold. No valves should ever be installed on ei the expansion or the overflow-lines, and in case the system is valved at the bo the expansion-line must be connected on the boiler-side of this valve; and wh two boilers are installed this line must be carried to a point above the wa line in the expansion-tank to prevent siphoning the water out of the en system in case it is necessary to drain only one boiler. Expansion or vent-connections must always be so made to main-piping in basement so that ai will be automatically removed from high points. Wherever possible rise branches to risers may be used for relieving any accumulation of air in the m piping. In SMALL INSTALLATIONS the expansion-line may be connected to return-riser of one of the highest radiators, and no overflow other than the need be provided for, as shown in Fig. 46. This is a cheap method, and sh not be resorted to unless extreme economy must be practised. The tank be in the same room with the radiator to prevent freezing, as no circulati provided for, and the overflow is simply discharged out of doors and us upon the roof. The usual result is that an unsightly appearance is soon cre

The United States Treasury Department employs a special vent and overf

) in cold climates, where there is liability of the vent-line freez-rough the roof, due to the condensation and freezing of vapor h this line. The vent-line is made only 2 ft high above the :pt within the building, and it is equipped with a check-valve ıpe of water through the same in case the tank should sud-he closing of the check-valve will compel the excess water to flow, and prevent the flooding of the building.

rstems. The PERMISSIBLE TEMPERATURES in any hot-water ›y the pressure on the system, which latter factor determines ›oiling will take place. The pressure at any elevation in an ɜr system will vary directly with the distance below the level ɜxpansion-tank, and hence it will be possible theoretically to he boiler at a temperature corresponding to the hydrostatic r before boiling would occur. The relation between hydro-and boiling-point are given in the following table:

ion between Hydrostatic Head, Pressure and Boiling-Point

0	12	24	37	49	61	74	87	100	113	125
0	5	10	15	20	25	30	35	40	45	50
212	227	239	250	259	268	274	281	287	292	298

l be quite impossible to carry temperatures in excess of 212° ı-tank system, as the high-temperature water would imme-›pen tank and boil. In order to overcome the limitations ›em, in which water will always boil as soon as a tempera-ached, various means of increasing the pressure in these sorted to in the attempt to carry a higher water-tempera-n very cold weather than would be possible with an open-devices have usually been installed on the expansion-line, r else just below the expansion-tank and the static head ing a column of mercury in the path of the expanding the expansion-tank.

the apparatus, known as the HONEYWELL HEAT-GENERA-44, in which it is seen that water entering the generator ›rce the mercury up the inner tube *A* until a head of 20 in at which time the entrance to this tube will be uncovered ater or air may enter it and pass to the expansion-tank. above that required to just fill tube *A* is returned by tube ıe base. When the system cools off water can flow back s the mercury-column drops in it, and the slight head of at the outlet of this tube is easily overcome by the head ›n-tank above this point. This increase of 10 lb in static sible to carry a maximum water-temperature of 240°, ı would be possible in an open-tank system. While a this could theoretically be carried AT THE BOILER in an a static head of 24 ft, just as soon as this water rose in the ınd escape from the expansion-tank, at the same time f water. In fact with the open-tank system the water t at a temperature of 212° F. The use of pressure-gene-bove makes it possible to use smaller radiators in the

heated rooms, as it is entirely possible to maintain steam-temperatures in t radiators whenever desired. Since higher temperatures are used, the differe between flow and return-riser temperatures becomes greater than in the op tank system, and hence a greater motive head exists and smaller mains a risers may be used with this system. The HONEYWELL COMPANY recomme the following schedule of radiator-tappings:

Table XXXVI. Riser-Sizes for Honeywell System

Pipe-size in inches	Capacity in square feet of hot-water radiation		
	1st floor	2nd floor	3rd floor
½	30	40	50
¾	75	100	125
1	75 up	100 up	125 up

It should be remembered that since radiators and pipes are smaller in t system there is much less water than in the open-tank system, making it m sensitive in warming up and also in cooling off. The GENERATOR should not placed close under the expansion-tank. Otherwise than this its location may anywhere in the expansion-line, as the same hydrostatic head is always act in addition to the head of mercury-column.

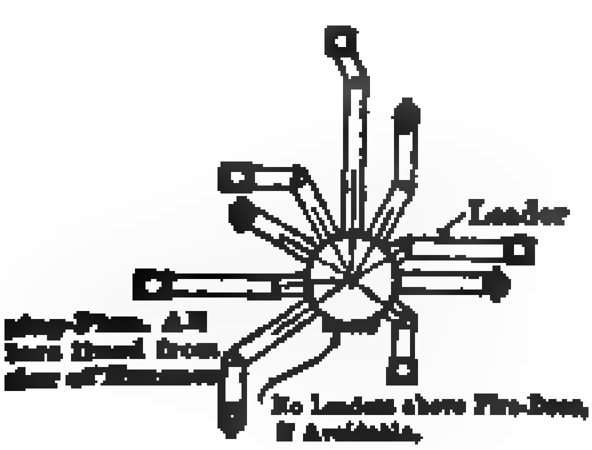
Furnace-Heating

The Furnace and Its Location. The method of warming or heatin building by what is generally known as a warm-air furnace is termed FURNA HEATING. The furnace consists briefly of a cast-iron or steel heater, contai a COMBUSTION-CHAMBER, FIRE-POT and GRATE. The heater is usually set a encased by a double-wall galvanized sheet steel JACKET (Fig. 47), although b is sometimes used instead of the steel jacket for this purpose. Furnaces soft coal are usually designed with a secondary air-supply or OVERDRAFT admitting heated air just at the surface of the fire in order to produce a m perfect combustion of the volatile combustible gases which are liberated fr this fuel immediately after firing. This overdraft should be under positive c trol so that it may be checked or closed after the fuel has been coked. Soft c may also be burned efficiently in the underfeed-type of furnace in which coal is from below by means of a plunger operating in a feed-chute discharging thro the center of the grate. The furnace should be located in the basement in approximately central position with reference to the rooms to be heated, a preferably toward the side or sides from which the prevailing winds blow in winter-time. This arrangement not only favors the more exposed rooms on floors above by shortening the leaders to these rooms, but also makes it poss to reduce the length of the cold-air duct, which should always be run from exposed side of the building to the cold-air pit below the furnace. (Figs. 53 54.) In operation cold air is drawn from the outside through the COLD DUCT, passed through the space between the heater and its jacket, and wa by coming in contact with the outside heated surface of the combustion-cha and the radiator, which is usuallly just above the combustion-chamber. I

discharged through flues connected at the top of the JACKET, FURNACE-CAB-
INET to the rooms to be warmed.

Fig. 47. Warm-air Furnace with Galvanized Sheet Steel Jacket

lers and Stacks. These connecting flues are made up of two sections, nearly horizontal round pipes in the basement, known as LEADERS (Figs. 49), which connect to the COLLARS on the top or conical sides of the



. Warm-air Furnace-leaders with Elbows

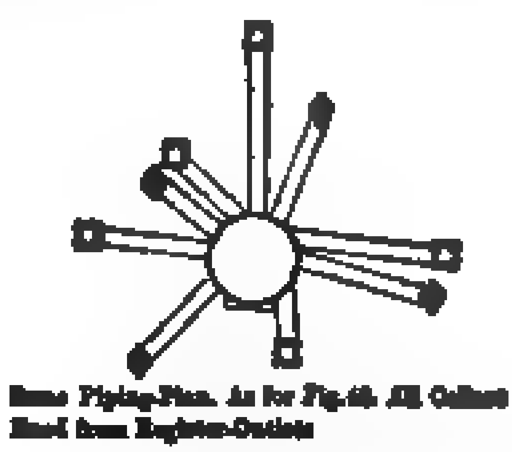


Fig. 49. Warm-air Furnace-leaders without Elbows

and (2) the vertical rectangular pipes called STACKS (Fig. 50), which he BOOT at the outer end of the leader, with the double-walled REGISTER- . 51) into which the REGISTER-GRILLE covering the opening into the room. The leaders should have an upward pitch toward the base of the

stack of at least 1 in per foot, and for the best results they should not be m than from 12 to 15 ft in length. The boots are made in a great variety of sha to suit actual conditions, and are simply ADAPTERS for the purpose of changi from round leaders to re angular stacks. The sta are usually run between t studding of interior walls partitions (Figs. 50 and 5 since if they are placed outside walls the cooling eff reduces their efficiency n only in temperature of a but also in velocity of fl The METAL used for lead and stacks is usually bri IX tin, although for lead larger than 12 in, galvani steel of No. 26 United Sta Standard gauge is usu employed. The covering all leaders, boots and stac as well as the furnace itsel most important, and eith heavy grade of asbestos pa is pasted on the outside, as in the case of leaders a the furnace itself, asbes air-cell covering, about ½ thick may be used and secu with brass bands or w Since the stacks must generally, in a 4-in studd space, with a net depth about 3¾ in, every ef must be made to keep th as deep as possible; and t lathing or expanded me should be used in front of such stacks, which ordina have only a single layer of asbestos-paper covering. A more effective insula may be provided by using a DOUBLE-WALL STACK, in which there is an air-s between the inside and outside pipes, and no asbestos covering is used. Table XLII for this equipment, as made by the Excelsior Steel Furn Company. Attention of the architect is here called to the fact that in the c of large second-floor rooms to be warmed by one register, 6-in stud partiti are generally required for the first floor.

Fig. 50. Vertical Stack with Side-wall Register

The Design of a Furnace Heating System.

Heat-Loss and Air Required. The determination of the size of the furn and the connecting leaders, stacks, registers, ducts, etc., is based on: (1) t actual heat-loss from each room in the building, including wall and glass-tra mission losses, as well as loss due to infiltration; and (2) the amount of air t circulated per hour, which in turn is based on this heat-loss. A buildin

by hot air by introducing the air into the rooms at a higher that required to be maintained in the rooms at the breathing-ly 70° F.) The air in cooling gives up per pound, 0.24 Btu r at con-'or each empera-ray sup-ssary to ansmis-tc., and rovides air for maxi-of the ter-cap 90° F., egisters a drop 15° be-bonnet These ximum eeded.

Fig. 61. Vertical Stack and Register-box

If the air is all drawn from the outside, and the re is 0°, then the air is heated from 0° to 190°, and the room, from 175° to 70°, or 105°. In other words, 0.24 s apparently thrown away, for every pound of air circulated. e brought in from the outside in order to supply a sufficient on, then this is the price which must be paid for ventilation, same, no matter what system of heating is employed, for tion. It is almost invariably the case, however, that a con-the air may, if desired, be recirculated, in which event, for ect, the furnace system of heating requires no more expendi-rm of fuel burned than a direct steam or hot-water system, as economical to operate when correctly designed, installed head producing the flow or circulation is due to the differ-en the ascending column of heated air and the weight of an the colder intake air The system may be proportioned f the air during the extreme cold weather.

be Circulated per Hour. It is first necessary to deter-r required per hour which must be supplied to each room.

's of air to be circulated per hour;
temperature to be maintained;
ature of air leaving the registers (assumed 15° lower than temperature leaving the furnace-cap or bonnet),
be supplied to room per hour as determined by heat-loss culations;
ven up per pound of air circulated.
$4(t_d - t)]$.
'or t_d is 175° F., and $t = 70°$ F.

of air, entering at temperature 175° = .063;
et of warm air entering room per hour
$= H/1.58$.

Heat Required from Heater per Hour, Based on Recirculation. Th heat required per hour from the heater will depend on the temperature of th entering air and will be a maximum when all the air circulated is taken fro the outside and a minimum when all of the air is recirculated.

Let h = Btu required from heater per hour;
t_e = temperature of air entering heater = 65°;
t_h = temperature of air leaving heater = 190°.

Then $h = 0.24\,W(t_h - t_e)$.

Substituting the values given above for W, t_h, and t_e,

$$h = 1.2\,H.$$

Size of Furnace. The capacity of a furnace for heating air depends primari upon the amount of coal that may be burned per hour, which is the product the RATE OF COMBUSTION by the grate-area. With an assumed or fixed rate combustion, the capacity of the furnace is dependent upon the grate-area. Th grate-area is therefore used as a basis for the rating and comparison of warm- furnaces. The average rate of combustion usual in furnace-heating rang from 3 to 4 lb per sq ft of grate-surface per hour, but in zero weather this ra may run as high as 6 lb, and is readily obtainable with the ordinary height residence-chimney; that is, at least 35 ft. A properly designed furnace w have a combined furnace and grate-efficiency of from 55 to 60%. Higher ef ciencies have been obtained in tests.

Commercial Ratings of Furnaces. Manufacturers rate their furna according to the amount of space, cubical contents, in the ordinary residen construction they will heat to 70° F. in zero weather. Maximum temperatu of air leaving registers = 175° F. The detailed dimension and capacity-da other than grate-area and space heated, of most furnaces are seldom publish by the manufacturer, although there are a few notable exceptions. The act size of the furnace naturally depends upon the heat-transmission of the wal floors and roofs, plus the infiltration-losses, as already explained. The cla however, is made that these "in turn bear a reasonably uniform relation to t cubical contents of the ordinary house," with the usual proportions and rat of wall to glass-surface, and that therefore the rating, as given, is justifia Tables XXXVII and XXXVIII were taken from the Warm Air Furnace Han

Table XXXVII. Capacity of Warm-Air Furnaces of Ordinary Constructio Cubic Feet of Space Heated

Divided space in cubic feet			Fire-pot		Undivided space in cubic fe		
+ 10°	0°	− 10°	Diameter, in	Area, sq ft	+ 10°	0°	−10°
12 000	10 000	8 000	18	1.8	17 000	14 000	12 00
14 000	12 000	10 000	20	2.2	22 000	17 000	14 00
17 000	14 000	12 000	22	2.6	26 000	22 000	17 00
22 000	18 000	14 000	24	3.1	30 000	26 000	22 00
26 000	22 000	18 000	26	3.7	35 000	30 000	26 00
30 000	26 000	22 000	28	4.3	40 000	35 000	30 00
35 000	30 000	26 000	30	4.9	50 000	40 000	35 00

XXXVIII. Air-Heating Capacity of Warm-Air Furnaces

...t	Casing *	Total cross-sectional area of heat-pipes, a, sq in	Number and size of heat-pipes that may be supplied
Area, sq ft	Diameter, in		
1.8	30–32	180	3–9″ or 4–8″
2.2	34–36	280	2–10 and 2–9″ 2–9″ and 2–8″
2.6	36–40	360	3–10″ and 2–9″ 4–9″ and 2–8″
3.1	40–44	470	3–10″ and 1–9″ 2–10″ and 5–8″
3.7	44–50	565	5–10″, 3–9″ 3–10″, 4–9″ and 2–8″
4.3	48–56	650	2–12″, 3–10″ and 3–9″ 5–10″, 3–9″ and 2–8″
4.9	52–60	730	3–12″, 3–10″ and 3–8″ 5–10″, 5–9″ and 1–8″

;-diameter should be such that the minimum cross-sectional area M, ; and radiator, will be at least 20% greater than the total cross-sectional heat-pipes, a, or $M = 1.2 \times a$ sq in.

ied by the Federal Furnace League, an association of United States ifacturers. This association is no longer in existence. If the he basement or leader-pipes exceed 12 ft in length or have less than e foot, or if more than one sixth of the outside surface of the building the furnace should be increased one or more sizes. The size of the iired can also be determined by the combined area of the cross- he warm-air pipes.

Rating Based on Efficiency and Rate of Combustion. The ir that a furnace is capable of imparting to the air (not the room) : estimated from the grate-area by assuming that the average coal intain approximately 12 000 Btu per lb. A combined furnace-and- ncy of 55%, and a maximum combustion-rate of 6 lb per sq ft of iour for maximum conditions (coldest weather) are also usually

urface Required, Based on Recirculation. The area of the dily calculated as soon as the heat to be supplied to the building per een determined.

H = Btu to be supplied building per hour;

h = Btu required from furnace per hour for heating the air = .12 H;

C = heating value Btu of coal per lb;

E = combined furnace-and-grate efficiency;

R = rate of combustion, pounds of coal per square foot of grate-surface per hr;

G = grate-area in square feet;

$h = G \times C \times E \times R = 1.2\,H.$

h = area of grate in square feet $\times$ 12 000 $\times$ 0.55 $\times$ 5.5
= 36 300 $\times G$. which is Btu transmitted to the air passing the furnace;

$G = (1.2 \times H)/36\,300.$

Table XXXIX. Dimensions and Capacities of the Kelsey Warm-Air Generator

Cold-air supply, etc.				Dimensions			
Size or number of generator	Inside diameter of brickwork if pit is used, in	Size of cold-air duct if air is taken from outside, in	Size of cold-air face if air is taken from inside, in	Diameter of base, in	Height of castings, in	Height of generator, cased complete, in	Measurements of galv. casings and tops, series A, B, C, in
[illegible]	34	12 × 24	18 × 24	38	49	61	35
[illegible]	38	12 × 30	20 × 26	42	53	63	40
[illegible]	42	12 × 36	21 × 29	46	56	68	43
[illegible]	49	14 × 40	24 × 32	53	59	69	50½
[illegible]	52	14 × 48	24 × 32	56	59	69	52
[illegible]	56	14 × 56 or (2) 14 × 32	30 × 36 or (2) 21 × 29	60	62	72	56½
[illegible]	60	14 × 60 to 72 or (2) 14 × 40	30 × 48 or (2) 24 × 32	64	66	76	59½

Size or number of generator	Heating capacities					Size of coal recommended
	House-heating			Church-heating		
	Number of average-size pipes or rooms	Estimated capacity, cu ft	Total area of heating-pipe supplied by each generator, sq in	Number of pipes	Estimated capacity, cu ft	
14	3 to 4	4 000–6 000	280 to 350	1	8 000	Chestnut
16	4 to 6	6 000–10 000	350 to 420	1 to 2	10 000–14 000	Chestnut or stove
18	6 to 8	12 000–16 000	450 to 500	1 to 2	16 000–20 000	Stove
21	9 to 11	18 000–24 000	575 to 625	1 to 2	25 000–35 000	Stove or egg
24	10 to 13	24 000–32 000	675 to 750	1 to 3	35 000–45 000	Stove or egg
27	12 to 15	32 000–42 000	850 to 925	1 to 3	50 000–60 000	Egg

he usual assumptions, with ANTHRACITE FUEL are:

$C = 12\,000$ Btu per lb;

$R = 4$ lb ordinary rate and 5.5 lb for maximum conditions in coldest weather;

$E = 0.55$;

hen $G = h/(12\,000 \times 5.5 \times 0.55) = h/36\,300 = 1.2H/36\,300.$

ize of Leaders and Stacks. The area of the air-pipes (leaders and stacks) uired for a room depends upon the quantity of air to be indroduced per min- and the velocity with which the air will flow with natural circulation.

$Q/60$ = cubic feet of warm air to be introduced into the room per minute;
V = velocity of air in feet per minute attainable;
H = heat-loss of room;
A = area of pipe in square feet;
$Q/60 = AV$, and substituting value of $Q = H/1.58$;
$A = H/(95 \times V)$.

e following velocities are approximately obtained in the leaders and stacks he floors as stated:

First floor, 175 ft per min;
Second floor, 240 ft per min;
Third floor, 310 ft per min.

e above velocities have been observed in practice in well-designed systems. for various floors, substituting in the above equation, in square inches:

$A_1 = H/115$ for first-floor pipes, leaders and stacks;
$A_2 = H/160$ for second-floor pipes, leaders and stacks;
$A_3 = H/206$ for third-floor pipes, leaders and stacks.

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ual leader and stack-sizes are based on the above areas, using the nearest ch for leader the diameter (Table XL), and keeping the stacks of such rtions that the cross-sectional dimensions are never in a greater ratio to 1. For example, a stack 4 by 20 in is seldom effective over its full area, s too narrow, and as its large rubbing-surface causes excessive friction. ctual velocities obtained, however, will depend upon the head or pressure g the flow and the friction-head, and will seldom exceed 50% of the theoret- elocities. Table XL has been recommended by the Federal Furnace and gives the sizes of round pipe for leaders, the size of wall-pipe for and free areas of registers to connect with same. Leaders over 12 ft in should be increased 1 in in diameter for each 5 ft beyond 12 ft.

isters. The FREE AREA through the ordinary register-grille is only ap- ately 55% of the gross area, and consequently a register must be selected s a gross area of double the area of the pipe with which it connects, in hat the air-passage may not be contracted and the capacity reduced. rcial register-sizes are based on the actual inside dimensions of the grilled and are made either of pressed steel or cast iron, with a variety of fancy grilles. The plain rectangular grille is to be preferred, finished to suit orative scheme, in black japan or electro-plated in brass, bronze or finish. Warm-air registers may be placed in the floor, but preferably e partitions, for first-floor rooms. By using the modern BASE-BOARD R, Fig. 52, it is usually possible to secure the required capacity without

Table XL. Capacities and Dimensions of Warm-Air Piping and Registers

Diameter of round cellar or riser-pipe, in *	Proper size of rectangular riser-pipe, in *	Area of riser-pipe, sq in	Required area of register-face, sq in *
6	3 × 9½	28	52
6½	3½ × 9½	33	62
7	3½ × 11	38	72
7½	3½ × 12½	44	84
8	3½ × 14	50	96
8½	4 × 14	57	108
9	4 × 16	64	120
9½	4 × 18	71	134
10	4 × 20	78	142
10½	6 × 14½	86	158
11	6 × 16	95	176
11½	6 × 17½	104	194
12	6 × 19	113	204
12½	6 × 20½	122	222
13	6 × 22	132	242
13½	8 × 18	143	254
14	8 × 19	154	276
14½	8 × 20½	165	298
15	8 × 22	176	320
16	8 × 25	201	358
17	10 × 22½	227	410
18	10 × 25½	254	450
19	12 × 23½	283	508
20	12 × 26	314	554
21	12 × 28½	346	618
22	14 × 27	380	686
23	14 × 29½	415	707
24	14 × 32	452	770

* When the required size of pipe falls on the odd half-inch (as 7½, 8½, 9½, etc.), the size may be increased to the even inch (as 8 instead of 7½, 9 instead of 8½, etc.), for the first-floor rooms and bath-rooms; provided that the pipes for upper-floor rooms other than bath-rooms, be decreased by ½ in when the required sizes fall on the odd half-inch. It is better, however, to use pipes of the sizes given in the above table, with proper allowances for length of pipe, extra bends, etc., beyond straight runs 12 ft long.

resorting to floor-registers. These base-board registers can be connected to a flue from 3 to 4½ in deeper than the studding. This has been accomplished by making the special base-board register so that it projects 2 in into the room at the floor-line, necessitating the cutting out of the floor, and also utilizing the space of about 1 in occupied by the lath and plaster, or a total increase in depth of flue of about 3 in. For upper-floor rooms registers should be placed in inside partition walls, using CONVEX REGISTERS for shallow stacks. As a general rule warm-air registers should be so placed as to shorten leader and stack-connections as much as possible. The use of a floor-register may be permitted in an entrance hall for drying shoes and garments, but it is unsanitary and cannot fail to collect dirt and filth of all kinds. In case such registers are used, however, suitable REGISTER-BOXES must be provided, and they are preferably constructed with double walls.

Example in Furnace-Heating. A gravity furnace-heating system is to be designed for the two-story frame building shown in Fig. 55, with inside and out

. Register
rat Floor
supply
las 7 in.
n. Deeper
ilding

Fig. 52. Modern Base-board Register

ig. 53. Cold-aid Ducts for Warm-air Furnaces

FRESH - AIR ROOM
WITH DUST-COLLECTOR

Table XLI. Table of Sizes of Floor-Registers, Base-Board Registers a[illegible] Register-Boxes

Size of round cellar-pipe, in	Size of round floor-register, in	Size of rectangular [illegible]	Size of register-box to base-board [illegible]	Size of base-board register [illegible]	Size of register-box to base-board register [illegible]	Size of base-board register where there is no limit to depth of register-box, in
6	9	8 × 8	2¾ × 10	7 × 10	2¾ × 10	7 × 10
6½	9	8 × 8	3¼ × 10	7 × 10	3¼ × 10	7 × 10
7	10	8 × 10	3¾ × 10	7 × 10	3¾ × 10	7 × 10
7½	12	8 × 12	4¼ × 10	7 × 10	4½ × 10	7 × 10
8	12	8 × 12	4½ × 12	7 × 12	4¼ × 12	7 × 12
8½	12	9 × 12	4¾ × 12	7 × 12	4¾ × 12	7 × 12
9	14	10 × 12	5 × 13	8 × 13	5 × 13	8 × 13
9½	14	10 × 14	6 × 12	10 × 12	6 × 12	10 × 12
10	14	10 × 16	6½ × 12	10 × 12	6½ × 12	10 × 12
10½	16	10 × 16	6½ × 13	10 × 13	6½ × 13	10 × 13
11	16	12 × 15	6¾ × 14	12 × 14	6¾ × 14	12 × 14
11½	16	12 × 18	7 × 15	12 × 15	7¼ × 14	12 × 14
12	18	12 × 20	6½ × 18	12 × 18	7½ × 15	12 × 15
12½	18	14 × 16	6¾ × 18	12 × 18	6¾ × 18	12 × 18
13	18	14 × 18	....	....	7¼ × 18	12 × 18
13½	18	14 × 20	....	....	8 × 18	12 × 18

Table XLII. Dimensions of Excelsior Double Wall-Pipe

Excelsior Steel Furnace Company

Number	Measurements: Nominal, in	Inside, in	Outside, in	Area of stack, sq in
4	3 ×10	2¾×10	3 ×10⅝	24
6	3 ×12	2¾×12	3 ×12⅝	28½
7	4 ×11	3 ×10	3⅝×10⅝	30
8	4 ×13	3 ×12	3⅝×12⅝	36
9	4 ×14	3 ×13	3⅝×13⅝	39
12	6 ×13	5 ×12	5⅝×12⅝	60
14	6½×14	5½×13	6⅝×13⅝	72

Number	Collar-diameter, in	Area of collar, sq in	Register-size, convex or wafer, sq in
4	7	39	6 × 8— 8 × 10
6	8 and 9	51 and 63	8 × 10— 9 × 12
7	8 and 9	61 and 53	8 × 10— 9 × 12
8	8, 9 and 10	51 and 78½	8 × 10—10 × 14
9	9 and 10	63 and 78½	10 × 12—10 × 14
12	9 and 10	63 and 78½	10 × 12—10 × 14
14	10 and 12	78½	10 × 14—12 × 14

of 70° and 0° F. respectively, and the air all recirculated in
·ansmission and infiltration-losses are as computed in Table

ig. 58. Furnace-heating Layout. (See data in Table XLVI)

ch also gives the size of heat-pipes, leaders and stacks, and register-

Furnace and Grate. The size of the furnace is calculated on the
ı that all the air is taken from the outside. The total calculated heat-
uilding per hour is 124 558 Btu, which, multiplied by 1 2, and divided

Table XLIII. Dimensions of Excelsior Single Furnace-Pipe

Excelsior Steel Furnace Company

Measurement in inches	Size of boot-collars, diameter, in	Capacity of collars, sq in	Capacity of pipe, sq in
3 ×10	8	51	30
3½×10	8	51	35
3 ×12	8 and 9	51 and 63	36
3½×12	9	63	42
3½×13	9 and 10	63 and 78	45
5½×12	10	78	66
5½×14	12	114	77
5½×16	12 and 14	114 and 154	88
7½×16	14	154	112

Note. Stacks 5½ in deep, made to order only. With frictionless boots, collars ame can be made with a diameter equal to width of stack. Collars 11 in in diamet urnished when so ordered.

Table XLIV. Capacities and Dimensions of Fresh-Air Ducts, Rooms, Etc.

Size of horizontal portion of rectangular fresh-air duct, in	Size of horizontal portion of round fresh-air duct, in	Cross-section area of horizontal portion of fresh-air duct, in	Size of fresh-air-room; length and width (height same as depth of cellar), in	Size of fresh-air intake (area of woven-wire netting, not including frame), in
8×18	1–14	144	18×48	12×16
8×21	1–15	168	21×48	14×16
8×24	1–16	192	24×48	16×16
10×21	1–16	210	21×60	14×20
10×24	1–18	240	24×60	16×20
10×27	2–13	270	27×60	18×20
10×30	2–14	300	30×60	20×20
12×27	2–14	324	27×72	18×24
12×30	2–15	360	30×72	20×24
12×33	2–16	396	33×72	22×24
12×36	2–17	432	36×72	24×24
12×39	2–17	468	39×72	24×26
14×36	2–18	504	36×84	24×28
14×39	2–19	546	39×84	26×28
14×42	2–19	588	42×84	28×28
14×45	2–20	630	45×84	28×30
14×48	2–21	672	48×84	28×32
14×51	2–21	714	51×84	28×34
16×48	2–22	768	48×96	32×32
16×51	2–23	816	51×96	32×34
16×54	2–24	864	54×96	32×36
16×57	2–24	912	57×96	32×38
16×60	2–25	960	60×96	32×40

ble XLV. Sizes and Capacities of Wooden Register-Faces for Cold-Air Ducts

Size	Net area of air-space, sq in	Nearest size of round pipe of equivalent area, in	Size, in	Net area of air-space, sq in	Nearest size of round pipe of equivalent area, in
2×20	135	12	24×24	323	20
2×24	161	14	24×26	349	20
2×30	202	16	24×30	403	22
4×20	157	14	28×28	439	22
4×26	203	16	30×30	504	26
6×20	179	14	36×20	403	22
6×24	215	16	36×24	484	24
6×30	269	18	36×30	605	28
8×24	242	18	36×36	725	30
8×30	303	20	72×18	726	....
0×20	224	16	72×20	806	....
0×24	269	18	72×24	968	....
0×26	291	18	72×30	1 210	....
×30	336	20	72×36	1 450	....

Table XLVI. Furnace-Heating Example (See Fig. 55)

First floor	Parlor	Hall	Dining-room	Library	Kitchen	
ic feet	2 280	2 170	2 400	2 280	3 600	Heat-loss of kitchen is based on kitchen-range supplying one-half the required amount.
t-loss, H, in tu per hour....	14 855	13 400	11 655	11 515	11 127	
a of heat-pipe, $\frac{H}{15}$ sq in......	127	116	101	100	96	
neter of leader inches	13	12	11	11	11	
of register in hes..........	12×18	12×18	12×15	12×15	12×15	

cond floor	Chamber No. 1	Chamber No. 2	Chamber No. 3	Chamber No. 4	Chamber No. 5	Bath-room
ents in cubic	2 052	1 458	1 746	1 206	1 242	576
-loss in Btu hour........	14 370	12 000	10 413	9 070	10 883	5 400
of heat-pipe, sq in........	89	75	65	56	68	34
eter of leader ches	10½	10	9	9	9	8
in inches...	5½×16	5½×14	5½×12	5½×12	5½×12	5½×10
f register in es *.........	12×14	10×13	10×12	9×12	10×12	10×12

net area of register-faces is assumed to be 55% of the gross area. The gros als 1.8 times the area of the leader-pipe.

by 36 300, the heat available from 1 sq ft of grate when burning 5.5 lb of coal of 12 000 Btu heat-value per pound, at 55% efficiency, gives 4.1 sq ft as the grate-area. This will require a grate of 28-in diameter. This building has a net volume of 26 000 cu ft, and by reference to Table XXXVII it is seen that a 28-in grate is recommended for this amount of divided space. The furnace, in this problem, has been located practically in the center of the house, but on the north side of its east and west axis, giving a direct cold-air connection from the north wall and short direct runs for most of the leaders.

Leader-Layout. The leaders may be laid off, as shown in Fig. 55 and in Fig. 48, by dividing up the circumference of the bonnet into areas proportional to the amount of air to be distributed by each leader, and then connecting collar and leader radially to furnace-cap, making one or more elbows in the leader, if necessary to connect with stack. Another method is to run practically all leaders direct from furnace to foot of stack (Fig. 49) and cut the collars in on the angles at which they intersect the casing. The former method is recommended and requires less skill in installation. The basement heating-plan is shown on the first-floor plan, which also shows all stack-sizes to both floors. Floor registers have been shown on the first-floor plan in order to simplify the layout and make the plan clearer. In general, base-board registers are to be preferred. The sum of the areas of leader-pipes is 927 sq in.

Hot-Blast Heating

General Features. The mechanical indirect method of heating, commonly known as the BLOWER SYSTEM OR HOT-BLAST SYSTEM, particularly adapted to the warming and ventilating of large structures, is made up of three units: (1) a HEATER constructed of pipes, tubes, or cast-iron sections, through which steam, hot water or hot gas may be passed. (2) A FAN OR BLOWER to circulate air over the heater-surfaces, the air acting as a heat-carrier or medium of heat-transfer. (3) A SYSTEM OF DUCTS OR PIPES to convey the heated air from the heater to points where heat may be required. When the heater is located between the fan and main duct, the combination is termed BLOW-THROUGH, and when the fan is installed between the heater and the duct, the arrangement is known as DRAW-THROUGH. These two arrangements are shown in Fig. 57. The DRAW-THROUGH combination is more often used for shop and factory-installations where compactness is desirable, the BLOW-THROUGH combination being used principally for HOT-AND-COLD systems as installed in schools and public buildings.

Advantages of the Blower or Hot-Blast System. The advantages of the blower or hot-blast system over those of direct radiation, briefly summarized are:

(1) When ventilation is a requirement in order to maintain a healthful atmosphere, this method affords a positive means of accomplishing this particularly desirable result, which is entirely independent of the changing climatic conditions.

(2) When a standard humidity of the air is to be maintained, a feature which is becoming to be more generally recognized as desirable in any heating-and ventilating installation, and quite essential to the successful manufacture of some materials, the humidifying-apparatus may readily be made an integral part of the system.

(3) A much smaller amount of radiating-surface is required to perform an equal heating-duty, with a consequent reduction in the number of steam-tight joints, unions and valves to keep in repair.

(4) The air-leakage being mostly outward, the building will in general be in

m drafts and more uniformly heated. If the air is simply recirculated, no ih air being taken into the heating system from the outside, the above statent does not apply. The pressure of the air in the building, even when all of air is taken into the heating system from the outside, is comparatively feeble, some air will enter by infiltration through the window and door-cracks on windward side of the building, although the statement is often made that leakage being all outward, prevents the infiltration of cold air from the iide.

) This system is more easily regulated, and readily responds to changing ide temperatures.

) The air entering for ventilation may be conveniently cooled in summer, r by the circulation through the heater of cold water or of brine previously d by mechanical refrigeration.

) Simply running the fan will in itself greatly relieve the oppressiveness in hot y weather, and when cold water is circulated through the coils the difference ry noticeable.

pical Arrangements. When ventilation is not a requirement, or when it is vely unimportant, as is frequently the case in shop or factory-heating where umber of persons vitiating the air is small compared with the cubical con- of the building, the air may be simply RECIRCULATED, sufficient fresh air ntilation being supplied by infiltration. The amount of heat to be sup- the heater in this case is the same as would be required for a direct-radiation lation. When ventilation is a requirement to be met a COLD-AIR INTAKE vided. Since the amount of air necessary for heating is generally in excess amount required for ventilation considerable economy may be effected by ılating a portion of the air. In this case only sufficient fresh air is drawn he system from the outside to meet the ventilation requirement and the ıder of the air necessary for heating, is recirculated. This may be readily d by an arrangement of ducts and dampers on the suction-side of the fan. fresh air introduced is to be washed or conditioned the washer or humid- ıd tempering-coil may be added between the inlet for the recirculated air e fresh-air intake.

ount of Air to be Circulated for Heating. The weight of air to be cir- l per hour for heating a room or building is found by dividing the heat-) by the amounts of heat given up by 1 lb of air in cooling from the tem- re at the duct-outlets to the mean room-temperature.

H = heat-loss of room, Btu per hr;
M = weight of air to be introduced in room per hour;
t = mean inside temperature;
t_d = temperature of air leaving duct-outlets.

$$M = H/[0.24(t_d - t)]$$

temperature t_d depends upon the temperature of the air entering the the velocity through the clear area, the amount of heating-surface and perature of the steam. This temperature in practice ordinarily ranges 5° to 150° F. and may be readily determined for any specified condition data given later under Hot-Blast Heaters. The temperature of the air the duct-outlets for ordinary installations, when the ducts are not run ound or in outside walls, may be assumed to be the same as the tempera- of the air leaving the heater. Any loss in temperature in this case vard heating the building and is therefore not a direct loss. If, how- e ducts are run underground or in outside walls, a considerable loss in

temperature may occur, which is a direct loss, and must be provided for by INCREASING THE TEMPERATURE OF THE AIR LEAVING THE HEATER by an amount equal to the estimated temperature-drop in the ducts.

Temperature of Air Entering Heater.

Let t_1 = temperature of air entering heater;

t_0 = outside temperature;

t = mean inside temperature;

t_2 = temperature of air leaving heater;

(a) When the air is all recirculated, $t_1 = t$;

(b) When fresh air only is circulated, $t_1 = t_0$;

(c) When a portion of the air is recirculated the resulting temperature of the mixture of fresh and recirculated air may be found by the METHOD OF MIXTURE

Let M_v = weight of fresh air, pounds required per hour for ventilation (30 cu ft per min per person);

= 0.075 × 1 800 × number of persons (usual requirements);

M_r = weight of air that may be recirculated;

$M = M_v + M_r$

$H = 0.24\,(M_v + M_r)\,(t_d - t)$.

Having assumed or fixed the value of t_d, the only unknown quantity is M_r.

$$M_r = H/[0.24(t_d - t)] - M_v$$

The temperature t_1 may then be found as follows:

$$M_v \times (t_0 + 460) = A$$
$$M_r \times (t + 460) = B$$
$$(M_v + M_r)\,(t_1 + 460) = A + B$$

or

$$t_1 = (A + B)/(M_v + M_r) - 460$$

Example. The heat-loss H for a certain factory-building is 70 600 Btu per hr. The number of men employed is 50. Mean inside temperature $t = 65°$ F. Outside temperature $t_0 = 0°$ F. Ventilation is to be provided at the rate of 1800 cu ft of fresh air per hour per person. Assumed temperature of air leaving duct-outlets is 135° F.

Solution. $M_v = 0.075 \times 1800 \times 50 = 6\,750$ lb per hr fresh air for ventilation. The weight of air that may be recirculated is

$$M_r = 706\,000/[0.24(135 - 65)] - 6\,750 = 35\,273 \text{ lb per hr}$$

The temperature of the air entering the heater will be:

$$6\,750 \times (\,0 + 460) = \;3\,105\,000$$
$$35\,273 \times (65 + 460) = 18\,516\,750$$

$$t_1 = (21\,621\,750/42\,023) - 460 = 55° \text{ F.}$$

If FRESH AIR ONLY is to be used, as in school-house and public-building heating, the weight of air to be circulated is determined directly by the ventilation requirement.

Then $M = M_v = H/[0.24(t_d - t)]$, or $t_d = (H + 0.24\,M_v t)/0.24\,M_v$

Temperature of Air at Duct-Outlets. When heating, by the hot-blast system, a building containing a number of rooms having different heat-loss and ventilation requirements, it is obviously impossible to maintain the de

perature by controlling the temperature (t_h) of the air leaving the heater at point. The temperature t_d of the air leaving the duct-outlets will ordinarily different for each room in the building, as shown in the following example. result is accomplished by the double plenum-chamber system described .

ample. Let it be required to determine the temperature of the entering t_d) to offset the heat-loss and provide ventilation for the several rooms as in Table XLVII. Inside temperature (t) to be maintained is 70°.

Table XLVII. Data for Example

oom-mber	Number of occupants, n	Ventilation		Heat-loss, H	Temperature of entering air, t_d (see formula)
		Cubic feet per hour at 70°, 1 800 × n	Weight per hour, M_0		
-1	50	90 000	6 750	32 000	89.5
-2	53	95 400	7 125	21 000	82.2
[all	....	30 000	2 250	4 000	77.4

perature of Air Leaving Heater. If all of the air is first warmed by the ING-COIL to 70° F., and a mixture of approximately (1 − x) parts of ed air and x parts of hot air is to be used, then the required temperature of air leaving the heater may be determined, for any particular case, by THOD OF MIXTURES previously given; or, assuming this temperature, the ions of hot and tempered air may be determined.

ple. Required the temperature of the hot air (t_h) leaving the heater for − 2, (Table XLVII) if the mixture entering the room is made up of tempered air at 70° and one half hot air. The total weight of air the room is 7 125 lb per hr, or 3562.5 lb of tempered air and 3562.5 lb r.

on. $3\,562.5 \times (70+460) + 3\,562.5 \times (t_h + 160) = 7\,125 \times (82.2 + 460)$.
$= 94$.

ing a temperature of $t_h = 120°$, it is required to determine the relative ons, by weight, of the mixture required.

= parts of hot air in mixture. Then $(1 - x)$ = parts of tempered air.

$$x(120 + 460) + (1 - x) \times (70 + 460) = (82.2 + 460)$$

$$x = 0.244 \text{ and } (1 - x) = 0.756$$

pplied for Ventilating Purposes Only. A combination of direct radia- ffset the heat-loss H, and a hot-blast system, to supply the fresh air r ventilation, is sometimes installed. In this case it is customary to eater of sufficient capacity to warm the air for ventilation to about 80°. r used for this purpose is made three sections deep and is often termed a G-COIL

d-Cold Systems. In order to accomplish the results required in the example, the so-called HOT-AND-COLD SYSTEM or DOUBLE-PLENUM-SYSTEM is used. All of the air drawn into the system from the outside

is first passed through a TEMPERING-COIL, which is designed to heat the air to approximately 70°. A portion of the tempered air is then passed through a heater and raised to 125° to 150°. Then if varying proportions of the hot and

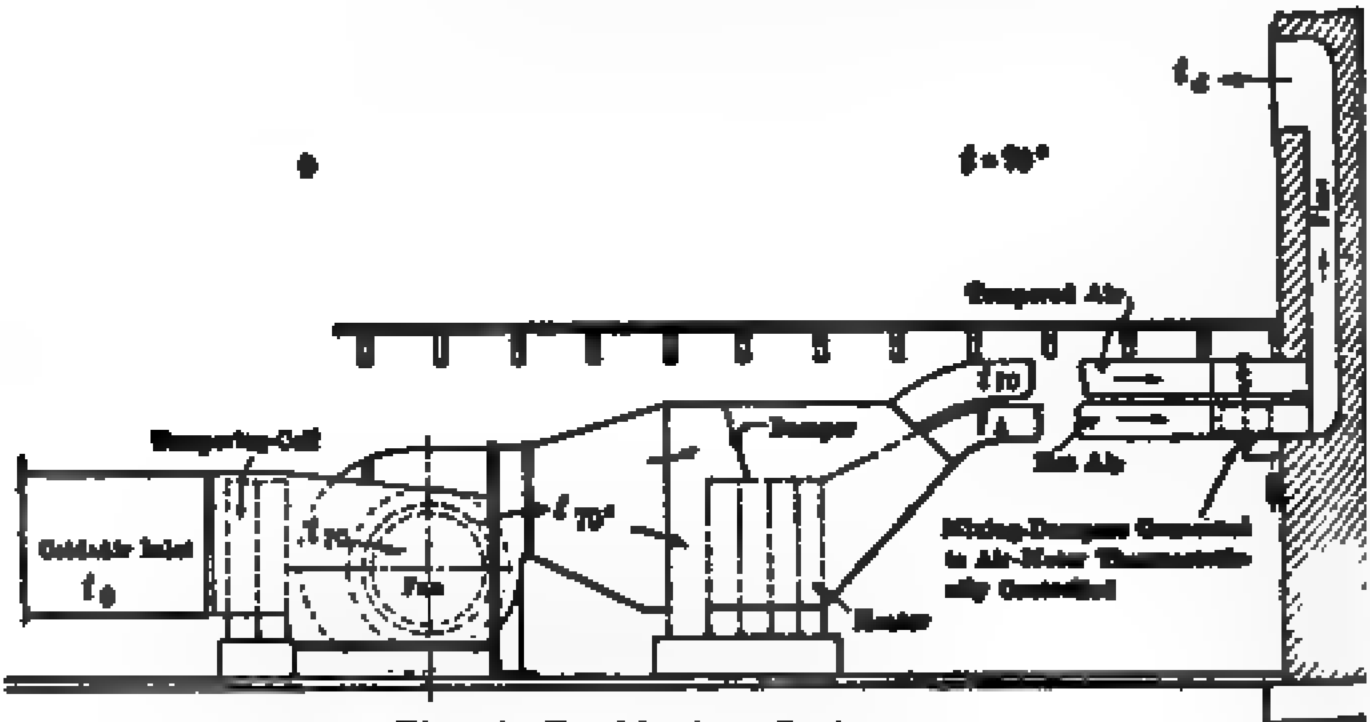


Fig. 58. Double-duct System

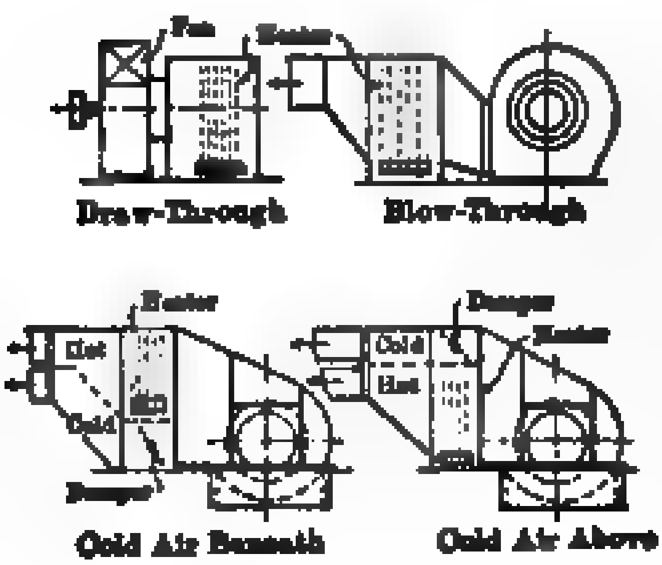


Fig. 57. Types of Heater-jackets

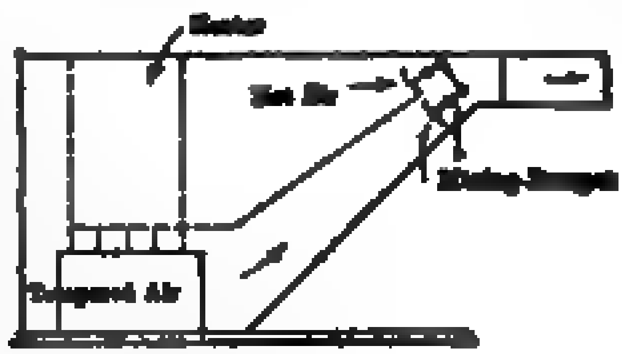


Fig. 56. Single-duct System

Fig. 59. Deflecting-damper for Branch-duct

Fig. 60. Thermostatically-controlled Mixing-damper

Figs. 56 to 60. Details of Ducts for Hot-and-cold Heating Systems

tempered air are correctly mixed the resulting temperature (t_g) is readily controlled without varying the quantity of air discharged, which evidently must remain constant on account of the ventilation requirement. There are two

thods of distribution used, as shown in Figs. 56 and 58. Referring to Fig. 58 seen that the hot and tempered air meet at the end of the plenum-chamber he entrance to the ducts, and the temperature of the mixture is controlled he MIXING-DAMPERS, which may either be hand-operated or placed under matic thermostatic control. It will be observed that the plenum-chamber vided, and that each duct serving a room has its own independent set of ng-dampers. This method of distribution is known as the SINGLE-DUCT EM and is frequently employed where the installation of the DOUBLE-DUCT EM, as described below, is not feasible or is undesirable. Fig. 56 shows a le set of ducts run from the plenum-chamber to the base of each vertical one carrying the hot air and the other the tempered air, the mixing being at the base of the flue as shown The mixing-dampers (Fig. 60) may be olled by hand by means of a chain carried up the flue and run into the at a point several feet above the floor-line, or placed under automatic mostatic control through the medium of a compressed-air-operated damper.

Hot-Blast Heaters

e-Coil Heaters. The type of heater that has been a standard for a er of years, for hot-blast work, is made up of four or eight vertical rows,

Fig. 61. Pipe-coil Heater-base

ng on the manufacturer, of 1-in pipe screwed into a cast-iron base and rom 2 1/8 to 2 3/4 in on centers, the pipes in each row being cross-connected op by the use of nipples and ells. The arrangement of coils and the of dividing the base by a vertical partition, running longitudinally, in separate the supply and return as used by the American Blower Com- shown in Fig. 61. A base with its accompanying pipes is termed a

SECTION. The heater is made up of a number of such sections enclosed b[y]
SHEET-STEEL JACKET, usually No. 22 gauge.

Cast-Iron Indirect Heaters. Special cast-iron sections for indirect hea[t]
are often used, and Fig. 62 shows a cast-iron heating-unit or section, na[m]
Vento, manufactured by the American Radiator Company, which is quite wi[d]
used in this class of work. A STACK made up of several sections has a sma[l]
number of joints than a pipe-coil section of equal heating-surface. The det[e]
oration of the cast-iron sectional type of heater is practically nothing except [i]
the right-hand and left-hand hexagonal nipples connecting the units which
to make up a stack. There are three standard lengths of Vento heater-secti[o]

Fig. 62. Plan of the Assembly of a Vento Heater

as indicated in Table XLVIII, which also includes other data required by t[he]
designer.

Table XLVIII. Vento Hot-Blast Heater-Data

Length of section in inches	Heating-surface, regular sections, in square feet	Free area in square feet, per section, inches on centers			Ratio, squa[re] feet heating-surface to fre[e] area for 5 in[.] on centers
		$4\frac{5}{8}$	5	$5\frac{3}{4}$	
40	10.75	0.52	0.62	0.72	17.34
50	13.50	0.65	0.77	0.91	17.53
60	16.00	0.78	0.92	1.08	17.39

Selection of Hot-Blast Heaters. General Conditions. In select[ing]
the size of a heater for any particular service the choice is based on the f[inal]
temperature desired and the FREE AREA required for a certain allowable velo[city].
That is, for any specified initial and final temperature desired, and a cer[tain]
number of sections, a final temperature results when the velocity has been fi[xed]
in advance. Good practice limits the velocity to the values given by the f[ol-]
lowing tables. High velocities are objectionable in public-building work,

ount of the resulting noise. The resistance through the heater increases roportion to the square of the velocity, which adds to the power required nove the air as the velocity is increased, as will be noted later.

ating of Hot-Blast Heaters. The RATING of an assembled heater of sev- sections (pipe-coil type) or stacks (Vento type) is based on the TEMPERATURE- of the air passing over the heating-surface for certain velocities through the or unobstructed area of the heater-face. The VELOCITY is based on the me of air at an assumed temperature of 70° for convenience in rating.

t M = weight of air to be circulated through heater per hour;
.075 = density of air at 70°;
A = free area of heater in square feet;
V = velocity of air in feet per minute through free area based on 70° temperature.

en

$$A = M/(60 \times 0.075 \times V) = M/4.5V \text{ sq ft.}$$

e following tables will serve as guides in selecting the VELOCITY V and the ER OF SECTIONS OR STACKS for various purposes.

Table XLIX. Allowable Velocities of Air through Vento Heaters

Referred to a temperature of 70° F.

Number of stacks deep, regular 5 in on centers	Public-building work, velocity in feet per minute	Factory work, velocity in feet per minute
4	1 000 to 1 500	1 200 to 1 600
5	1 000 to 1 300	1 200 to 1 600
6	1 000 to 1 200	1 200 to 1 600
7	900 to 1 100	1 200 to 1 500
8	800 to 1 000	1 200 to 1 400

L. Number of Heater-Sections for Pipe-Coil Heaters or Stacks of Vento Ordinarily Required

Service	Number of sections or stacks	Number of rows of 1-in pipe
c buildings, fresh air, exhaust-steam..........	5	20
trial buildings, fresh air, 4-lb gauge..........	6	24
trial buildings, fresh air, exhaust-steam.......	7	28
trial buildings, recirculation, 5-lb steam......	5	20
ering-coils, fresh air, exhaust-steam..........	·3	12

perature-Rise. The TEMPERATURE-RISE of the air passing through hot- eaters of various types has been well established by experiment, the manufacturers having published the results in the form of bulletins and es.

ple. It is required to determine the size of a Vento hot-blast heater to he necessary heat for a public building, the calculated heat-loss of which

is $H = 1\,420\,000$ Btu per hr for 70° inside and 0° outside temperature. Th temperature of the air entering the rooms, t_d, is to be approximately 120°. Th steam-pressure is 5-lb gauge. The temperature of the air entering the heat is 0°.

Solution. First determine from Table LI the number of stacks deep require for F.T. at 120° and entering air at 0°, using a velocity of 1000 ft per mi This condition calls for a 5-stack-deep heater. Then determine the weight the air to be circulated per hour.

$$M = H/[0.24(t_d - t)] = 1\,420\,000/[0.24(120 - 70)] = 118\,333 \text{ lb}$$

The free area required is

$$A = 118\,333/(4.5 \times 1\,000) = 25.1 \text{ sq ft}$$

Table LI. Final Temperatures and Condensations, Vento Heaters

Regular section, Standard spacing, 5-in center to center, of loops. Steam, 5-lb gaug C is the condensation in pounds per hour per square foot of heating-surface. F. is the final temperature of air leaving heater.

Number of stacks deep	Temperature of entering air	Velocity through heater in feet per minute, measured at 70°							
		1 000		1 200		1 400		1 600	
		F.T.	C.	F.T.	C.	F.T.	C.	F.T.	C.
1	20	51	1.99	49	2.23	47	2.42	45	2.56
	30	60	1.92	58	2.17	56	2.33	54	2.46
	40	68	1.80	66	2.00	64	2.16	62	2.26
	60	84	1.54	82	1.69	81	1.89	80	2.05
	70	92	1.41	90	1.54	89	1.71	88	1.85
2	20	76	1.80	72	2.00	69	2.20	66	2.36
	30	83	1.70	79	1.89	76	2.06	73	2.21
	40	90	1.60	86	1.77	83	1.93	81	2.10
	60	103	1.38	100	1.54	98	1.71	96	1.85
	70	110	1.28	107	1.42	105	1.57	103	1.69
3	20	97	1.65	92	1.85	88	2.06	85	2.22
	30	103	1.56	98	1.75	94	1.91	91	2.08
	40	109	1.47	104	1.64	100	1.79	97	1.95
	60	120	1.28	116	1.44	113	1.58	110	1.71
	70	126	1.20	122	1.34	119	1.46	116	1.57
4	20	115	1.52	110	1.73	105	1.91	101	2.0[illegible]
	30	120	1.44	115	1.63	110	1.80	106	1.95
	40	124	1.35	119	1.52	115	1.68	111	1.8[illegible]
	60	134	1.19	129	1.33	125	1.46	122	1.5[illegible]
	70	138	1.09	134	1.23	131	1.37	128	1.4[illegible]
5	20	130	1.41	124	1.60	119	1.78	114	1.9[illegible]
	30	134	1.33	128	1.51	123	1.67	118	1.8[illegible]
	40	138	1.26	132	1.42	127	1.56	123	1.7[illegible]
	60	145	1.09	140	1.23	136	1.36	133	1.5[illegible]
	70	149	1.01	144	1.14	141	1.27	138	1.[illegible]
6	20	142	1.30	136	1.49	130	1.65	126	1.[illegible]
	30	145	1.23	139	1.40	134	1.56	130	1.7[illegible]
	40	148	1.15	143	1.32	138	1.47	134	1.[illegible]
	60	155	1.02	150	1.15	146	1.29	142	1.[illegible]
	70	...	....	...	....	...	....	...	....
7	20	152	1.21	146	1.39	141	1.55	136	1.7[illegible]
	30	155	1.15	149	1.31	144	1.46	139	1.6[illegible]
	40	158	1.08	153	1.24	148	1.39	143	1.5[illegible]

ble LI (Continued). Final Temperatures and Condensations, Vento Heaters

		Velocity through heater in feet per minute, measured at 70°							
mber of icks eep	Temperature of entering air	1 000		1 200		1 400		1 600	
		F.T.	C.	F.T.	C.	F.T.	C.	F.T.	C.
1	−20	...	....	...	....	...	....	...	....
	−10	...	....	...	....	...	....	...	....
	0	35	2.24	32	2.46	...	....	...	....
2	−20	49	2.22	44	2.46	40	2.69	37	2.92
	−10	56	2.12	51	2.35	47	2.56	44	2.77
	0	62	1.99	58	2.23	54	2.42	51	2.62
3	−20	75	2.03	69	2.28	64	2.51	59	2.70
	−10	80	1.92	75	2.18	70	2.39	66	2.60
	0	86	1.84	81	2.08	76	2.27	72	2.46
4	−20	96	1.86	90	2.12	84	2.34	78	2.51
	−10	101	1.78	95	2.02	89	2.22	84	2.41
	0	106	1.70	100	1.92	95	2.13	90	2.31
	−20	114	1.72	107	1.95	100	2.15	94	2.34
	−10	118	1.64	111	1.86	105	2.06	99	2.24
	0	122	1.56	115	1.77	109	1.96	104	2.14
	−20	129	1.59	121	1.81	115	2.02	110	2.22
	−10	132	1.52	125	1.73	119	1.93	114	2.12
	0	135	1.44	129	1.65	123	1.84	118	2.02
	−20	141	1.47	134	1.69	128	1.90	122	2.08
	−10	144	1.41	137	1.62	131	1.81	126	1.99
	0	147	1.35	140	1.54	135	1.73	130	1.90
	−20	151	1.37	144	1.58	138	1.77	133	1.96
	−10	153	1.31	147	1.51	141	1.69	136	1.87
	0	156	1.25	150	1.44	144	1.62	139	1.78

ing to Table XLVIII and choosing a 60-in length of units, 5 in on centers, und that the free area per section is 0.92 sq ft. The number of sections d across the face of the heater is

$$A/0.92, \text{ or } 25.1/0.92 = 27$$

ating-surface per section is 16 sq ft. The total heating-surface is there-

$$S = 5 \times 27 \times 16 = 2\,160 \text{ sq ft}$$

condensation per hour is

$$2\,160 \times 1.56 \text{ (Table LI)} = 3\,370 \text{ lb}$$

pplied by the boiler, or by exhaust-steam, at 5-lb pressure.

Design of Air-Ducts

ure-Loss. The frictional resistance of air flowing through smooth tal ducts, commonly termed PRESSURE-LOSS, measured in inches of r 70° air and for a length of duct equal to 100 ft, is given by the follow-ula:

$$h = 0.000136 \times (R/A) \times v^2$$

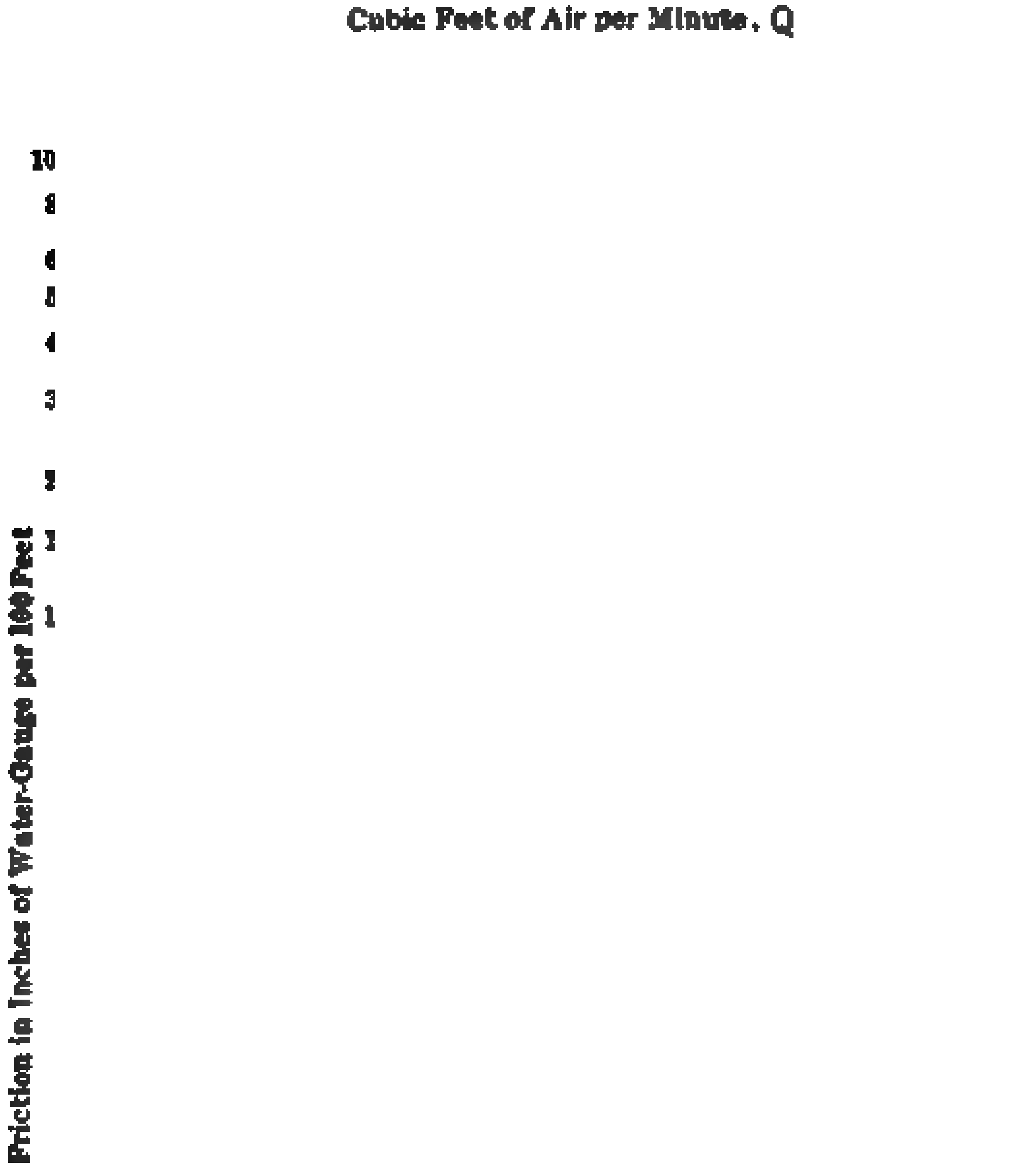
R is the perimeter of the duct in feet, *A* the area of duct in square e velocity of the air in feet per second, and *h* the pressure-loss

measured in inches of water-column. For round ducts the above form reduces to

$$h = 0.00055v^2/D$$

in which D is the diameter of the duct in feet.

The diagrams in Figs. 63 and 64 are based on this formula, from which t



METER OF A ROUND DUCT for various velocities, and the PRESSURE-LOSS or ISTANCE for various quantities of air flowing, may be found without solving above equation.

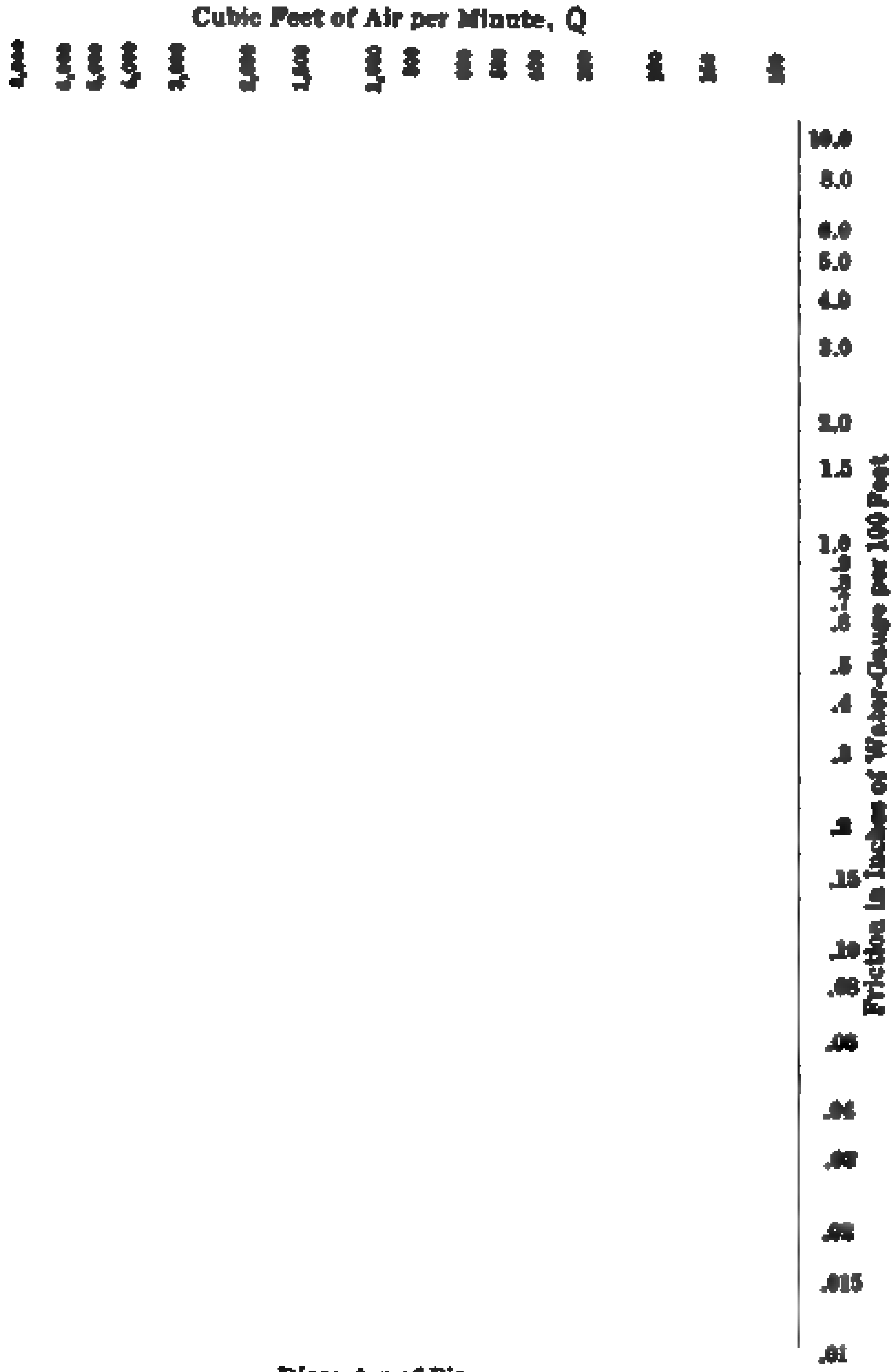


Fig. 64. Diagram of Friction-pressure Loss

ple. What should be the size of a round duct required to convey 1 500 air per minute with a velocity of 1 800 ft per min; and what is the pres- per 100 ft of duct.

Solution. Locate 1 500 on the upper side of the pipe-diagram in Fig. 64, an pass horizontally downward until the 1 800-ft-velocity diagonal line is inte sected. The duct which comes nearest to the required size has a diameter 12 in. At this intersection pass to the right side to the base-line and read 0.4 in water-pressure loss.

Allowable Velocity of Air in Ducts and Flues. In order to limit the resis ance or pressure-loss in the duct system the designer should, in general, keep t velocities within the limits stated in Table LII. In public-building work t air should be delivered to a room at a velocity that will insure its moveme to the desired points in the room without objectionable draft or noise in passi through the register-grills.

Table LII. Allowable Velocities in Hot-Blast Systems

Types of buildings	Allowable velocity in fee per minute
Public buildings	
Through free area of wall-registers	400– 500
Through free area of floor-registers	200– 300
Vertical flues to registers	600– 750
Connections to base of flues	800–1 000
Main horizontal distributing ducts	1 500–2 500
Manufacturing plants	
In plants where the occupation is more or less sedentary and the employe sits all day feeding automatic machinery:	
Main ducts	1 200–1 500
Branches	600– 900
In plants where the employe stands all day, as in machine-shops, foundries, etc.:	
Main ducts	1 500–2 400
Branches	900–1 500

The velocity through the fan-outlet, under the ordinary conditions that obtain in heati work, varies from 1 500 to 2 500 ft per min.

Table LIII. Metal Gauges for Ducts

American Blower Company

Heating and ventilating		Thickness and weight		Blowpiping and exhaust work	
Diameter in inches	United States standard gauge-number	Thickness in inches and weight in pounds per square foot		Diameter in inches	United States standard gauge-number
		in	lb per sq ft		
6–18	26	0.1087	0.91	3– 5	26
19–36	24	0.025	1.16	6– 8	24
38–48	22	0.0312	1.41	9–15	22
50–60	20	0.0375	1.66	16–24	20
63–72	18	0.05	2.16	26–30	18

. Pipes and Ducts. The recommended gauge (United States ge) for various sizes of galvanized sheet-steel pipes for heating work, blowpiping and exhaust work, is given in Table LIII.

ss of Rectangular Ducts. The simplest method of determining rtion the system for ROUND DUCTS throughout, and then transfer R SIZES giving equal pressure-losses (not equal areas) by means of

V. Round and Rectangular Ducts of Equal Pressure-Losses

4	6	8	10	12	14	15	16	18	20	22	24
Equivalent diameters in inches											
4.4	..	..	..	..	..	..	..	..	..	..	..
4.9	..	..	..	..	..	..	..	..	..	..	..
5.4	6.6	..	..	..	..	..	..	..	..	..	..
5.8	7.0	..	..	..	..	..	..	..	..	..	..
6.1	7.6	8.8	..	..	..	..	..	..	..	..	..
6.5	8.0	9.3	..	..	..	..	..	..	..	..	..
6.8	8.4	9.8	11.0	..	..	..	..	..	..	..	..
7.1	8.8	10.2	11.5	..	..	..	..	..	..	..	..
7.4	9.2	10.7	12.0	13.2	..	..	..	..	..	..	..
7.6	9.6	11.1	12.5	13.7	..	..	..	..	..	..	..
7.6	9.9	11.5	12.9	14.3	15.4	..	..	..	..	..	..
8.2	10.2	11.9	13.4	14.7	16.0	16.5	..	..	..	..	..
8.4	10.5	12.3	13.8	15.2	16.5	17.1	17.6	..	..	..	..
8.6	10.8	12.6	14.2	15.7	17.0	17.6	18.2	..	..	..	..
8.9	11.1	13.0	14.6	16.1	17.4	18.1	18.7	19.8	..	..	..
9.1	11.4	13.3	15.0	16.5	17.9	18.6	19.2	20.4	..	..	..
9.3	11.6	13.6	15.4	17.0	18.4	19.0	19.7	20.9	22.0	..	..
9.7	12.1	14.2	16.1	17.8	19.2	19.9	20.6	21.9	23.1	24.2	..
10.0	12.6	14.8	16.8	18.5	20.0	20.8	21.5	22.8	24.0	25.2	26.4
10.4	13.1	15.4	17.3	19.2	20.8	21.6	22.3	23.8	25.1	26.3	27.5
10.8	13.5	15.9	18.0	19.8	21.5	22.4	23.1	24.6	26.0	27.3	28.5
11.0	13.9	16.4	18.5	20.5	22.2	23.1	23.9	25.4	26.8	28.2	29.5
11.3	14.3	16.9	19.1	21.1	22.9	23.8	24.6	26.2	27.7	29.1	30.5
11.6	14.7	17.3	19.6	21.6	23.5	24.4	26.3	26.9	28.5	30.0	31.3
11.9	15.1	17.7	20.1	22.2	24.2	25.1	26.0	27.7	29.3	30.8	32.2
12.2	15.4	18.2	20.6	22.8	24.8	25.8	26.7	28.4	30.0	31.5	33.1
12.5	15.7	18.6	21.1	23.3	25.4	26.4	27.3	29.1	30.8	32.4	33.9
12.7	16.1	19.0	21.6	23.8	25.9	26.9	27.9	29.8	31.4	33.0	34.5
13.0	16.4	19.4	22.0	24.3	26.5	27.5	28.5	30.3	31.2	33.7	35.3
13.3	16.7	19.8	22.4	24.8	27.0	28.1	29.1	31.0	32.8	34.6	36.2
13.5	17.0	20.1	22.8	25.2	27.5	28.6	29.6	31.6	33.4	35.2	37.0
13.7	17.3	20.4	23.2	25.7	28.0	29.2	30.3	32.2	34.1	35.9	37.6
13.9	17.6	20.8	23.6	26.2	28.5	29.6	30.7	32.9	34.7	36.5	38.3
14.1	17.9	21.1	24.0	26.6	29.0	30.1	31.2	33.4	35.3	37.2	38.9
14.3	18.2	21.5	24.4	27.0	29.5	30.6	31.7	33.9	35.9	37.8	39.6
14.6	18.4	21.8	24.7	27.4	30.0	31.1	32.2	34.4	36.4	38.4	40.3
14.7	18.7	22.1	25.1	27.8	30.5	31.6	32.7	34.9	37.1	39.1	40.9
15.0	19.0	22.4	25.5	28.2	30.9	32.1	33.2	35.4	37.7	39.6	41.6
15.1	19.2	22.7	25.9	28.6	31.3	32.6	33.7	35.9	38.2	40.2	42.2
15.3	19.5	23.0	26.2	29.0	31.7	33.0	34.2	36.4	38.7	40.8	42.8
15.5	19.7	23.3	26.5	29.4	32.1	33.4	34.7	36.9	39.2	41.4	43.4

Example. What is the width of a rectangular duct 6 in high equivalent to the pressure-loss for a duct 12-in in diameter? **Solution.** 22 in.

Table LV. Friction Pressure-Loss of 90° Elbows

Radius of throat in diameters of pipe	1/4	1/3	3/4	1	1 1/4	1 1/2	2	3	4	5
Number of diameters of straight pipe of equivalent pressure-loss	67	30	16	10	7.5	6	4.3	4.8	5.2	5.8

Example. A duct 12 in in diameter and 120 ft long contains two 90° elbows. The ratio of the radius of throat to pipe-diameter is 3. The amount of air flowing is 1 500 cu ft per min and the velocity 1 800 ft per min.

Solution. The total equivalent length of duct is

$$120 + (2 \times 4.8) = 129.6 \text{ ft}$$

The pressure-loss, from the diagram of Fig. 64, is 0.48 in per 100 ft. The loss

$$0.48 \times (129.6/100) = 0.62 \text{ in of water}$$

The pressure-loss through register-grills may be taken at 0.023 in for a velocity of 400 ft per min through free area. The gross area of registers is twice the free area. The pressure-loss in standard air-washers for a velocity of 400 ft per min through the free area may be assumed to be 0.15 in of water. In the case of humidifiers, in which the spray is directed against the flow of air, a pressure-loss of 0.55 in of water may be assumed for preliminary estimates. The values assumed for this loss vary with different manufacturers. The pressure-loss through hot-blast heaters may be taken from Table LVI.

Table LVI. Friction of Air through Vento Heaters

Friction-loss, in inches of water, due to air passing through Vento stacks. Regular section. Standard 5-in spacing of loops. Air-temperature 70° F.

Velocity in feet per minute	One stack	Two stacks	Three stacks	Four stacks	Five stacks	Six stacks	Seven stacks
800	0.037	0.070	0.103	0.135	0.167	0.200	0.23[illegible]
900	0.047	0.088	0.129	0.170	0.211	0.252	0.293
1 000	0.059	0.109	0.160	0.211	0.262	0.313	0.364
1 100	0.071	0.132	0.193	0.255	0.316	0.377	0.438
1 200	0.084	0.157	0.230	0.303	0.376	0.449	0.522
1 300	0.099	0.185	0.271	0.356	0.442	0.528	0.614
1 400	0.115	0.214	0.314	0.414	0.513	0.612	0.712
1 500	0.132	0.246	0.360	0.474	0.588	0.702	0.81[illegible]
1 600	0.150	0.280	0.410	0.540	0.670	0.800	0.93[illegible]
1 700	0.169	0.316	0.463	0.609	0.756	0.903	1.0[illegible]
1 800	0.190	0.354	0.518	0.683	0.848	1.012	1.17[illegible]

Effect of Temperature on Pressure-Losses. The preceding data on pressure-losses in ducts, registers and heaters are based on an air-temperature of 7[illegible]

other temperatures, the pressure-losses are to be multiplied by the ratio, ity of air at actual temperature to density at 70°. These ratios are given 'able LVII. For heaters use the average temperature of the air passing ugh the heater.

e LVII. Ratios of Density of Air at Actual Temperature to Density at 70° F.

emperature	Factor	Temperature	Factor
100	0.945	140	0.880
120	0.910	150	0.865
130	0.890	160	0.850

sign of Duct Systems. There are two schemes used in PROPORTIONING UCTS: (1) the velocity method, and (2) the method of equal friction pres- oss per foot of length. The first method involves the fixing of the veloci- see Table LII) in the various sections, and the gradual reduction of the ty from the beginning of duct to the point of discharge. In this case the re-loss is computed separately for each section having a different velocity ie various pressure-losses added together to obtain the total loss in pres- The second method is used principally in the design of duct systems for y-heating. The velocity in the outlet farthest from the fan is fixed and ea and diameter of this branch are determined by the volume of air to vered. The friction pressure-loss per 100 ft of a duct of this size is deter- by the diagrams in Figs. 63 and 64. The remainder of the main duct is roportioned for this same pressure-loss per 100 ft.

mple. The first method is illustrated in Fig. 65, showing a single-duct . The risers are figured for a velocity of 600 ft per min, or 10 ft per sec;

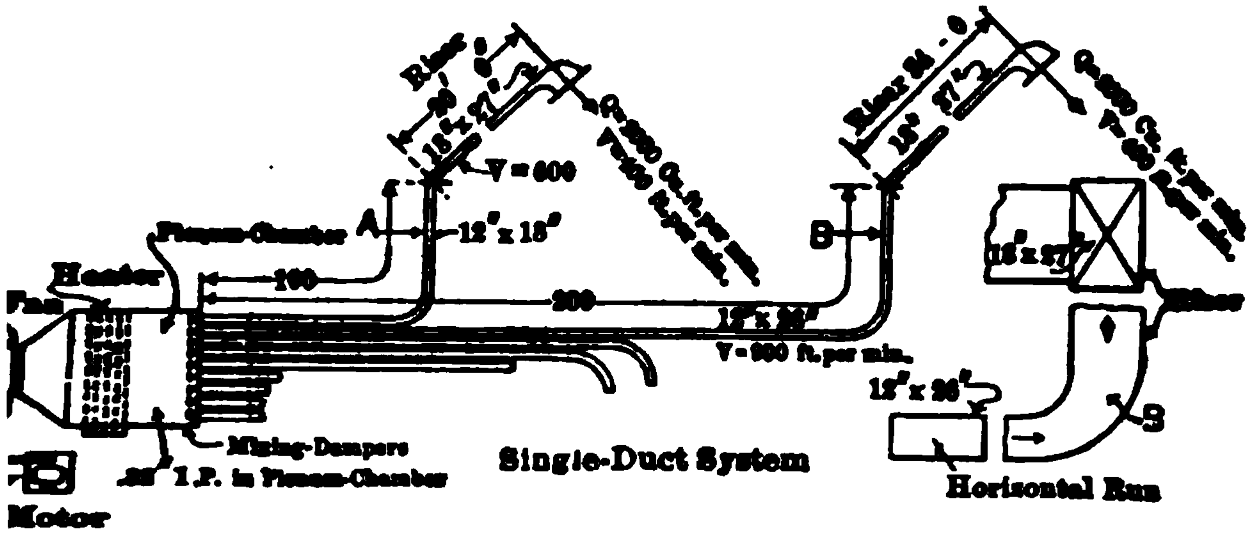


Fig. 65. Single-duct System

ft per min, or 6.6 ft per sec; through free area of register-grill. The in the longest main, *B*, is 900 ft per min, the volume of air to be delivered ft per min, and the temperature 120°.

n. The area of the riser required is

$$2\,000/600 = 3\tfrac{1}{3} \text{ sq ft, or } 480 \text{ sq in}$$

27-in riser, giving 486 sq in area, is used. The area of main, *B*, is

$$2\,000/900 = 2.22 \text{ sq ft, or } 320 \text{ sq in}$$

The size of the duct is 12 by 27 in. The diameter of a round duct for the sa
friction-loss for the riser, from Table LIV, is 24 in, and for the main, B, 19.5

Referring to the diagram in Fig. 64, the pressure-loss for the riser is 0.032
per 100 ft. For the main the pressure-loss is 0.09 in per 100 ft. The main h
one elbow and the riser one elbow, and the ratio of radius at throat to diame

Fig. 66. Part of Basement-layout for a Single-duct System for a School-building

will be assumed to be 3, in both cases. The equivalent length of main, B
therefore

$$200 + (4.8 \times 20/12) = 208 \text{ ft}$$

and the pressure-loss is

$$0.09 \times (208/100) = 0.187 \text{ in}$$

The equivalent length of riser is

$$34 + (4.8 \times 24/12) = 44 \text{ ft}$$

and the pressure-loss is

$$0.032 \times (44/100) = 0.014 \text{ in}$$

'he pressure-loss through the register-grill is 0.023 in. The total resistance he duct system is therefore

$$0.0187 + 0.014 + 0.023 = 0.224 \text{ in}$$

ssuming that a five-section Vento heater is employed, with a velocity (fig-l through free area) of 1 200 ft per min, the pressure-loss through the heater .376 in (Table LVI). The total resistance against which the fan must ate is

$$0.224 \text{ in} + 0.376 \text{ in} = 0.60 \text{ in}$$

d on 70° air. Assuming the temperature of the air to be 120°, the resist-is

$$0.60 \times 0.91 \text{ (Table LVII)} = 0.55 \text{ in}$$

e second method of duct-design is illustrated by the example given under ication of Hot-Blast Heating-Data, page 1342.

Ventilating Fans

el-Plate Fan. The standard type of fan that has been used for a number of in hot-blast work is known as the STEEL-PLATE FAN, the construction of the being shown in Fig. 67. As the name implies, the wheel and casing of this re constructed of steel plate ght structural sections, the having eight to twelve , straight or slightly curved periphery, and in a direc-opposite to the rotation. late fans are designated by r, this number being the imate height of the fan-in inches.

ibladed Fan. A new type , known as the SIROCCO or TURBINE-TYPE IMPEL-g. 68, has recently entered d of heating and ventilat-l on account of its higher y, quieter running, and ly smaller size for the same than the steel-plate fan, dly supplanting the latter. her efficiency is accounted for by the material reduction of the air-e or pressure-head loss by friction through the fan, due to the shorter nd the larger inlet, which is of practically the same diameter as the self. This fan deserves more than passing mention as it represents the greatest single improvement ever made in the design of a centrif-

Fig. 67. Standard Steel-plate Fan-wheel

g of Fans. The volume of air at 70° which a fan will deliver (cubic feet te) varies with the resistance against which it operates. In order to fan from Table LIX, the resistance (static pressure) must first be ed by the duct-design, and after the size of the heater has been chosen esistance determined. The speed and brake horse-power required to

drive the fan are also stated in the tables. The temperature of the air hand by the fan with DRAW-THROUGH apparatus is higher than 70°, except for a f which is connected ahead of a tempering-coil, usually a two-section-deep heat The tabulated speed, volume and brake horse-power to maintain the press must be multiplied by the fact given in Table LVIII for temperatu other than 70°. The above fact in this table are the square roots the ratios of the density of the ai 70° F. to its density at the tempe ture stated.

Fig. 68. Sirocco Wheel or Turbine-type Impeller

Table LVIII. Factors for Speed, V ume and Brake Horse-Power

Temperature in degrees Fahrenheit	Factors
0	0.932
100	1.028
120	1.046
130	1 055
140	1.064
150	1.073

Application of Hot-Blast Heating-Data

The Application of the Foregoing Data on hot-blast heating to a facto building follows (see Fig. 69). The calculated heat-loss is 1 423 920 Btu per

Conditions. Air recirculated and inside temperature maintained at 6 Velocity of air through heater, from 1 000 to 1 200 ft per min. Velocity of at last outlet in duct system, 1 275 ft per min. Temperature of air deliv by heater, 145°.

Weight of Air to be Circulated per Minute. This is

$$1\,423\,920/[0.24(145 - 60)160] = 1\,163 \text{ lb}$$

Size of Heater. This condition, 60° initial and 145° final tempera requires a heater five stacks deep (Table LI), and a velocity of 1 000 ft per through free area at a temperature of 70°. The volume of air per m measured at 70°, to be handled by the heater and fan is

$$1\,163/0.075 = 15\,506 \text{ cu ft}$$

The FREE AREA required is:

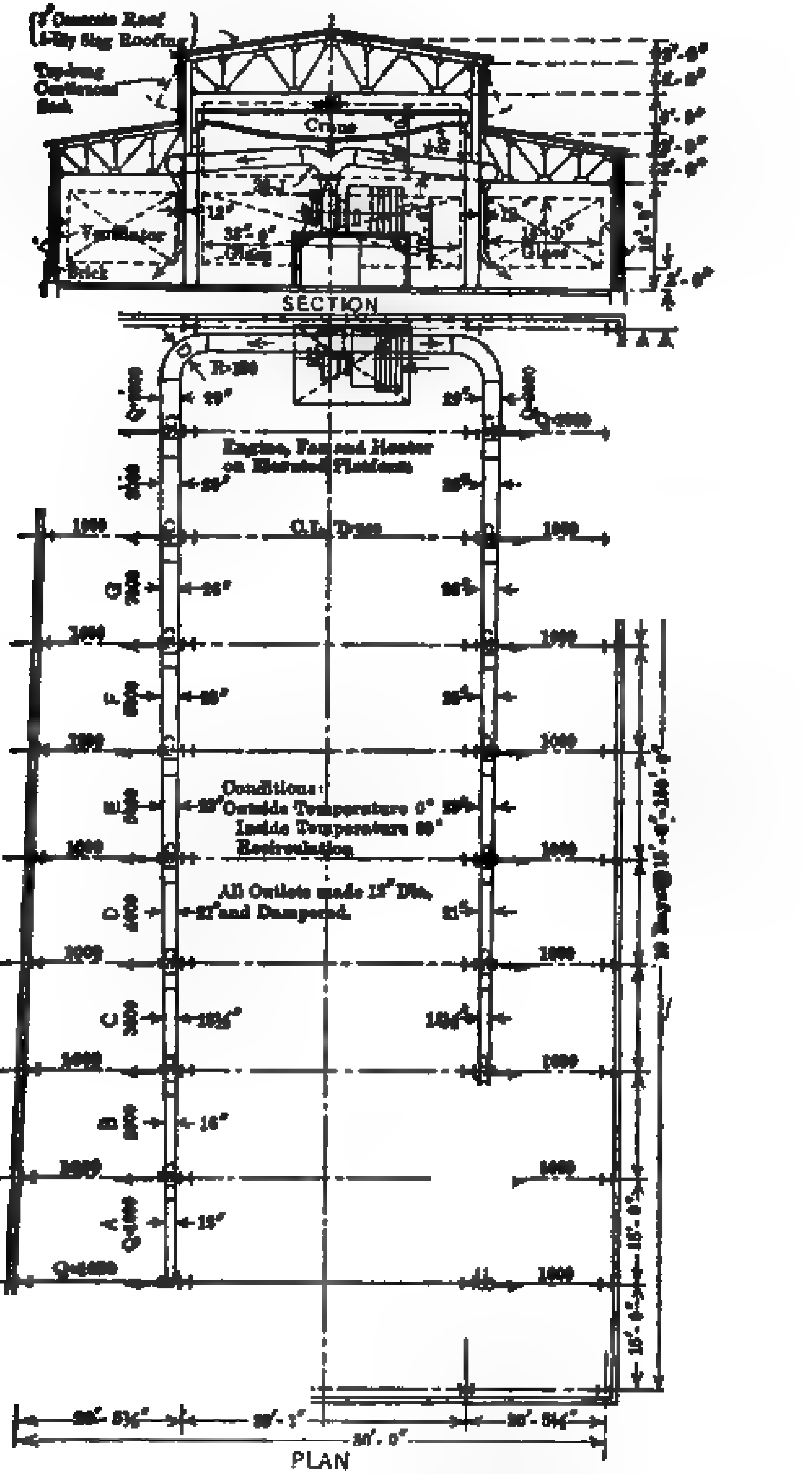
$$15\,506/1\,000 = 15.5 \text{ sq ft}$$

Assuming a 60-in length of section with loops 5 in on centers, the free are section is 0.92 sq ft (Table XLVIII). The number of sections per stack is

$$15.5/0.92 = 17$$

The total number of sections required is

$$5 \times 17 = 85$$



Hot-blast Heating for a Factory-building. (See Example for Computations)

Table LIX. Speeds, Capacities and Horse-Powers, American Sirocco Single-Inlet, Standard-Width Fans

American Blower Company

Air-temperature, 70° F. Ratio of total to static pressure, 1.15. Ratio of velocity to static pressure, 0.15. Fifty-per-cent opening

an- nb	neter of eel in iches	Maintained resistance or static pressure in inches of water-gauge	¼	½	¾	1	1¼	1½	2
4	24	C. F. M.*	425	3 480	4 260	4 920	5 500	6 020	6 9
		R. P. M.	285	320	391	453	505	554	6
		B. H. P.	.244	.526	.97	1.485	2.08	2.725	4.
4½	27	C. F. M.	070	4 410	5 400	6 205	6 970	7 620	8 8
		R. P. M.	200	284	348	402	449	492	5
		B. H. P.	.308	.664	1.215	1 86	2 62	3.44	5.
5	30	C. F. M.	790	5 450	6 650	7 690	8 600	9 416	10 8
		R. P. M.	180	256	313	362	403	443	5
		B. H. P.	.378	.811	1 492	2.295	3.225	4.22	6.
6	36	C. F. M.	460	7 835	9 580	11 060	12 350	13 540	15 6
		R. P. M.	150	214	260	302	336	369	4
		B. H. P.	.541	1.06	2.14	3.28	4.6	6.01	9.
7	42	C. F. M.	425	10 670	3 050	15 070	16 800	18 425	21 2
		R. P. M.	129	183	223	259	288	316	3
		B. H. P.	.731	1.57	2.9	4 44	62 2	8 15	12
8	48	C. F. M.	720	13 920	7 000	19 700	22 000	24 100	27 8
		R. P. M.	112	160	196	226	252	277	3
		B. H. P.	.954	2 045	3.76	5.78	8.14	10.61	16.
9	54	C. F. M.	350	17 615	1 500	14 860	27 800	30 440	35 1
		R. P. M.	100	142	174	201	224	246	2
		B. H. P.	.206	2.58	4.73	7.26	10.22	13.38	20
.0	60	C. F. M.	150	11 760	6 500	10 750	34 300	37 650	43 4
		R. P. M.	90	128	156	181	202	222	3
		B. H. P.	.473	3.17	5.81	8.95	12 55	16 48	25

								[illegible]	30.45
12	72	C. F. M.	21 800	31 350	38 300	44 240	49 400	54 130	62 500
		R. P. M.	75	107	130	151	168	185	214
		B. H. P.	2.11	4.54	8.35	12.81	18 0	23 55	36 2
13	78	C. F. M.	25 600	36 800	45 000	52 000	58 100	63 600	73 500
		R. P. M.	69	99	120	140	155	171	197
		B. H. P.	2.465	5.31	9.8	15.0	21.1	27 6	42.5
14	84	C. F. M.	29 650	42 650	52 100	60 200	67 300	73 700	85 000
		R. P. M.	64	92	112	130	144	158	183
		B. H. P.	2.87	6.15	11 33	17.4	24.45	32 0	49.2
15	90	C. F. M.	34 100	49 000	59 900	69 230	77 500	84 700	97 800
		R. P. M.	60	86	104	121	135	148	171
		B. H. P.	3 27	7.06	13.0	19 93	28 0	36 6	56.4
16	96	C. F. M.	38 800	55 900	68 100	78 750	88 400	96 500	111 800
		R. P. M.	56	80	98	112	126	139	160
		B. H. P.	3.73	8.0	14.8	22 65	31.8	41.6	65.1
17	102	C. F. M.	43 800	63 100	77 000	89 000	99 900	109 000	126 000
		R. P. M.	53	75	92	106	119	130	150
		B. H. P	4.21	9.03	16.75	25.6	35.9	46.9	73.5
18	108	C. F. M.	49 200	70 750	86 400	99 900	12 000	122 000	141 500
		R. P. M.	50	71	87	100	112	123	142
		B. H. P.	4 72	10.15	18.8	28.65	40.3	52.6	82.5
19	114	C. F. M.	54 750	78 750	96 200	111 000	24 500	136 000	157 200
		R. P. M.	47	67	83	95	106	117	134
		B. H. P.	5 25	11.3	20.9	31.9	45 0	58.6	91 8
20	120	C F. M	60 600	87 300	06 800	123 000	138 000	150 800	174 500
		R. P. M.	45	64	78	91	101	111	128
		B. H. P.	5 82	12.5	23 1	35 4	49 7	65.0	101.8

Double-inlet fans have approximately double the capacities of single-inlet fans and require for their operation, under the same conditions, approximately twice the power. The "per-cent opening" refers to the opening used for the rating given from the test-data.

* C. F. M. denotes cubic feet per minute, R. P. M., revolutions per minute and B. H. P., brake horse power.

The total heating-surface is

$$85 \times 16 = 1\,360 \text{ sq ft}$$

Weight of Steam or Condensation per Hour. This is

$$1\,360 \times 1.09 \text{ (Table LI)} = 1\,482 \text{ lb}$$

The equivalent amount of direct radiation is

$$1\,482/0.25 = 5\,929 \text{ sq ft}$$

Design of Duct System. The round ducts will be designed for equal-frictio pressure-loss per foot of length. The final velocity at the last or most remot outlet from fan will be taken at 1275 ft per min. The friction pressure-loss fo this velocity, as read from the diagram in Fig. 64, is 0.25 in of water per 100 of length. There are to be eighteen outlets. The total volume of air to be di charged, measured at 145° F., is

$$1\,163/0.065 = 18\,000 \text{ cu ft per min}$$

or

$$18\,000/18 = 1\,000 \text{ cu ft per min per outlet}$$

The cross-sectional area of the outlet or last section is 1 000/1 275 sq ft, co responding to a circular section with a diameter of 12 in. The branch-outle may all be made the same size and provided with dampers to adjust or equali the flow. The friction pressure-loss in the duct system is therefore

$$(212/100) \times 0.25 = 0.53 \text{ in of water}$$

The size of each section of duct is determined by locating the quantity of a at the right of the diagram and passing horizontally to the intersection with th 0.25-in pressure-loss line.

Table LX. Data for Design of Ducts in Fig. [illegible]

Section	Quantity of air in cubic feet per minute 145° F.	Duct-diameter in inches	Velocity in feet per minute	Measured length plus allowance for ells, in feet
A	1 000	12	1 275	25 + [1 × (6 + 3)] = 34
B	2 000	16	....	15
C	3 000	18½	....	15
D	4 000	21	....	15
E	5 000	23	....	15
F	6 000	25	....	15
G	7 000	26	....	15
H	8 000	28	....	15
I	9 000	29	....	35 + [2.4(6 + 10)] = 7
J	18 000	38	2 285	....
				Total length = 212

Selection of Fan for Draw-Through Arrangement. The static press rating required, referred to a temperature of 70°, is:

Pressure-loss in heater (data from Table LVI) = 0.26 in
Pressure-loss in duct (data from chart, Fig. 64) = 0.53 in

Total = 0.79 in

The actual pressure-loss will be somewhat less, owing to the fact that the air-
mperature is higher (145° F.) and the density less than for air at 70° F. The
tual estimated pressure-loss is therefore assumed to be ¾ in.

The volume of air the fan must handle in this example is 18 000 cu ft per min,
easured at 145° F. As stated under Rating of Fans, to maintain a constant
essure the tabulated speed, volume and horse-power must be multiplied by
e square root of the ratio of densities, or

$$\sqrt{0.075/0.066} = 1.07 \text{ (nearly) (Table II)}$$

We therefore select from Table LIX a fan having a capacity, measured at
' F., equal to

$$18\,000/1.07 = 16\,822 \text{ cu ft per min (approximately 17 000)}$$

en operating with a static pressure of ¾ in. A No. 8 Sirocco fan fulfills this
uirement. The tabulated speed and horse-power when multiplied by the
or 1.07 gives

$$196 \times 1.07 = 210 \text{ R.P.M.}$$

$$3.76 \times 1.07 = 4.02 \text{ brake horse-power}$$

election of Fan for Blow-Through Arrangement. In this case the
may be called upon to handle air at a temperature of 0° F., or lower. Assum-
he same weight of air, or 70 000 lb per hr, to be handled by the fan at a static-
sure of ¾ in, the volume at 0° is

$$70\,000/(0.086 \times 60) = 13\,566 \text{ cu ft per min}$$

rring to Table LVIII, the ratio between the speed, volume and power neces-
to produce the same pressure for air at 0° and air at 70°, is found to be
. We therefore choose a fan with a capacity of

$$13\,566/0.932 = 14\,557 \text{ cu ft of air at } 70°$$

with a static pressure of ¾ in.

m-Engine. When high-pressure steam is available an automatic high-
engine is frequently employed for fan-driving, and the exhaust from the
e is used in the first section of the heater.

ection of Motor for Fan-Driving. It is considered good practice to add
10 to 15% to the brake horse-power, as determined from the fan-tables,
e rating of the motor, to allow for a possible overload due to the fact that
n may not be operated under exactly the same conditions as to pressure
eed as those under which it was originally rated. For the preceding exam-
raw-Through Arrangement) a 5-horse-power motor would be selected.

litional Heating Requirement. It is frequently desirable to proportion
ating-apparatus large enough so that the fan may be shut down at night
arted up about two hours before the shop or factory is opened in the
g. In this event it may be safely assumed that the temperature of the
the building will not be below 30° F. when the fan is started, and that
is all recirculated. The fan and heater must be of sufficient capacity
e care of the heat-loss from the building, including the infiltration, and
tion to warm up the contained air from 30° to 60° in two hours. Assum-
same data as given in the preceding example, the additional heat re-
will be, if the cubic contents of the building are 328 000 cu ft,

$$(328\,000 \times 0.08 \times 0.24 \times 30)/2 = 94\,464 \text{ Btu per hr}$$

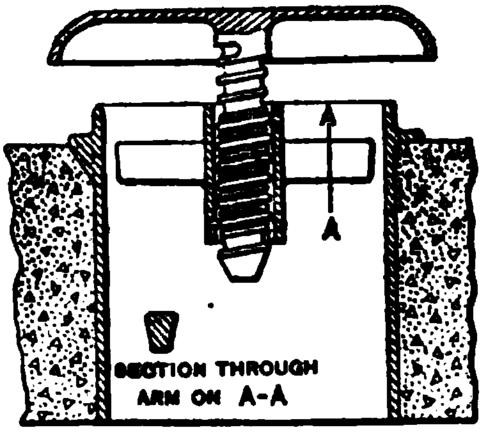
This amounts to an increase of approximately 7% in the heating requirements as previously calculated, and is readily provided for by increasing the steam pressure carried in the heater to approximately 10-lb gauge. Catalogues, bulletins, etc., on the subject of hot-blast heating, air-washing and humidification may be obtained from the American Blower Company, the B. F. Sturtevant Company, the Buffalo Forge Company, and the Carrier Air Conditioning Company.

Ventilation

Natural and Mechanical Ventilation. Ventilation, whether NATURAL or MECHANICAL, consists in the displacement of vitiated air from an apartment and

s replacement by fresh air. To state that the air in an apartment is renewed ny given number of times per hour is not strictly accurate, as a positive change oes not actually occur; the incoming air mixes with and dilutes the foul air to a oint suitable for healthful respiration. In NATURAL-VENTILATION systems the iovement of the air in flues, ducts, etc., is induced solely by the thermal head roduced by the difference between the density of the column of air in the ucts and that of the outside atmosphere; the higher the temperature in the ucts the more powerful the draft. The direction and velocity of the wind aterially affect the natural ventilation, retarding or accelerating the moveent of the air through ducts and flues, according to the exposure of the building id the position of inlets and outlets. In MECHANICAL VENTILATION the moveent of air is maintained by means of various types of fans, driven by a steamgine, electric motor, or other prime mover. With fans of known efficiencies e results can be accurately estimated. The principal advantages of the use mechanical systems of heating and ventilation have already been stated der Hot-Blast Heating.

Systems of Ventilation. Ventilation systems are also broadly divided into o general classes known as the UPWARD SYSTEM and the DOWNWARD SYSTEM. e UPWARD SYSTEM (Fig. 70) is generally used for audience-rooms where there is ong natural tendency for the heat given off by the large number of occupants to e and take with it the vitiation-products due to respiration. The AIR IS SUPED NEAR THE FLOOR-LINE through mushroom ventilators in the floor, or ough the hollow pedestals of the chairs themselves, or through low registers. e VITIATED-AIR OUTLETS ARE IN OR NEAR THE CEILING. This system makes it her difficult to heat the room in advance of the arrival of the audience as the lets allow the warmed air to escape almost rapidly as it can be introduced. The VNWARD SYSTEM is very generally used chool-rooms, hospitals, institutions, etc. occupants are not as closely placed as he former case, and a more even distrion of air and more uniform heating can ecured when the AIR IS SUPPLIED EIGHT T OR MORE ABOVE THE FLOOR, and the ATED AIR REMOVED AT OR NEAR THE OR-LINE. On account of the elevation e inlets above the heads of the occupants e is little liability of drafts, and if the ts are on the same side wall as the inthere is very little opportunity for -circuiting between inlet and outlet, the incoming air must flow out across oom to the cold outside wall before it cool and drop to the floor-level. It is, ver, necessary in the downward system, vercome the natural tendency of the d air from the bodies of the occupants to rise and oppose the uniform ward tendency of the incoming fresh air. The selection of either system depend entirely on the conditions to be met. These have been outlined above paragraphs.



SIZES OF "ABC" MUSHROOM VENTILATOR.

Size.	Approximate Inside Diameter.	Approximate Weight
4	4¼	6 lb.
5	5¼	10 lb.
6	6¼	15 lb.

Fig. 71. Section through A B C Mushroom Ventilator

stribution of the Air. In general, it should be observed that whether rd or downward ventilation is employed there should always be a definite n of vitiated-air removal, designed to provide for uniform distribution and

prevent short-circuiting between inlets and outlets. A practically complete diffusion can only be attained when inlet and outlet are placed in the same inside wall, with the former at least from 7 to 8 ft above the latter. MULTIPLE

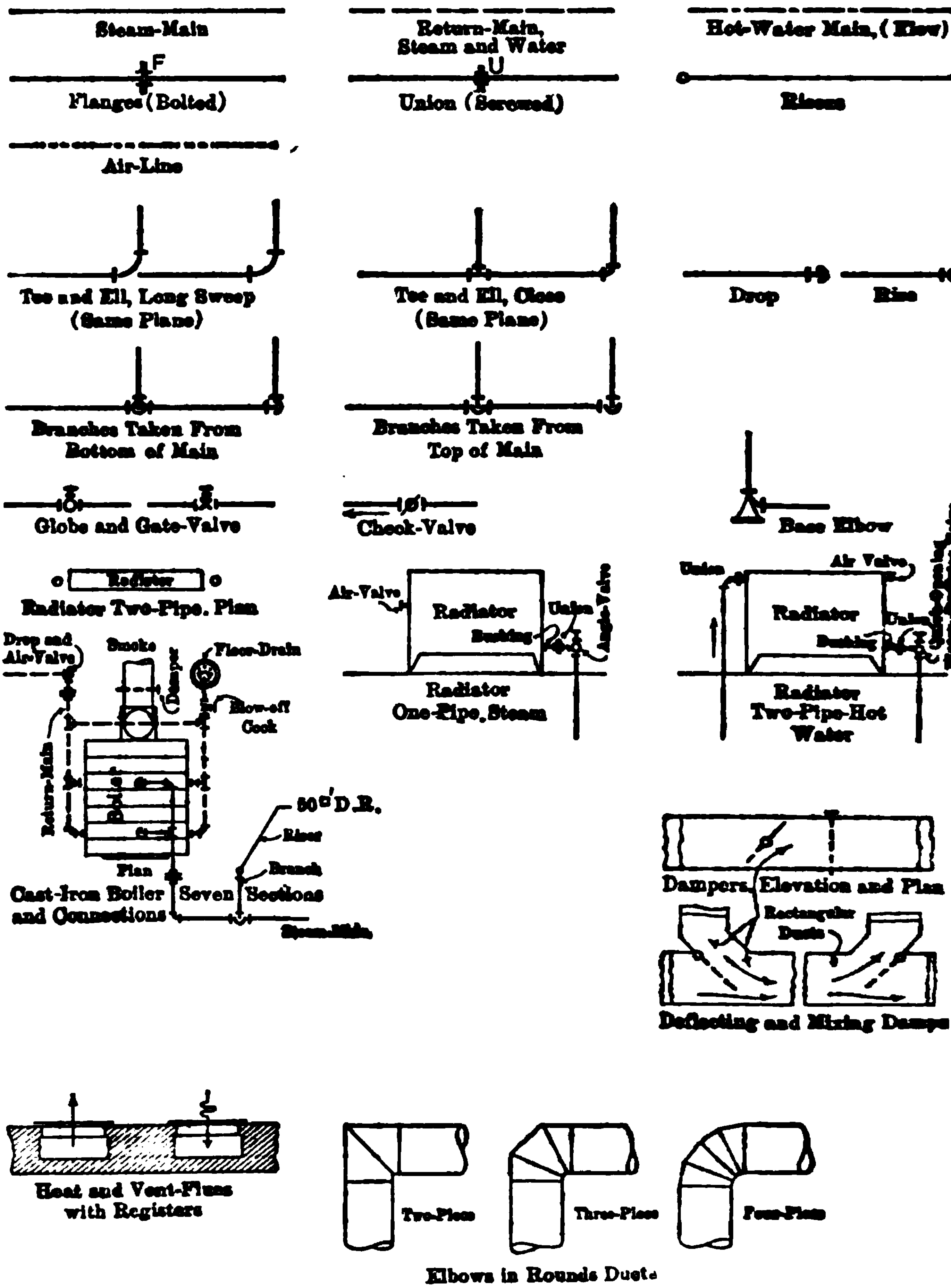


Fig. 72. Heating and Ventilating Symbols

INLETS and MUSHROOM VENTILATORS, in order to secure a better mechanical distribution of the air, are being made use of in many systems of upward ventilation for audience-rooms with fixed seats. In this case a false floor or PLENUM CHAMBER must be constructed just below the main floor through which the air

to be supplied. Mushroom ventilator-heads (Fig. 71) are then located under ery second or third seat and adjusted to give a uniform discharge of tempered r over the entire seating-area. These heads are either mounted on an ad-

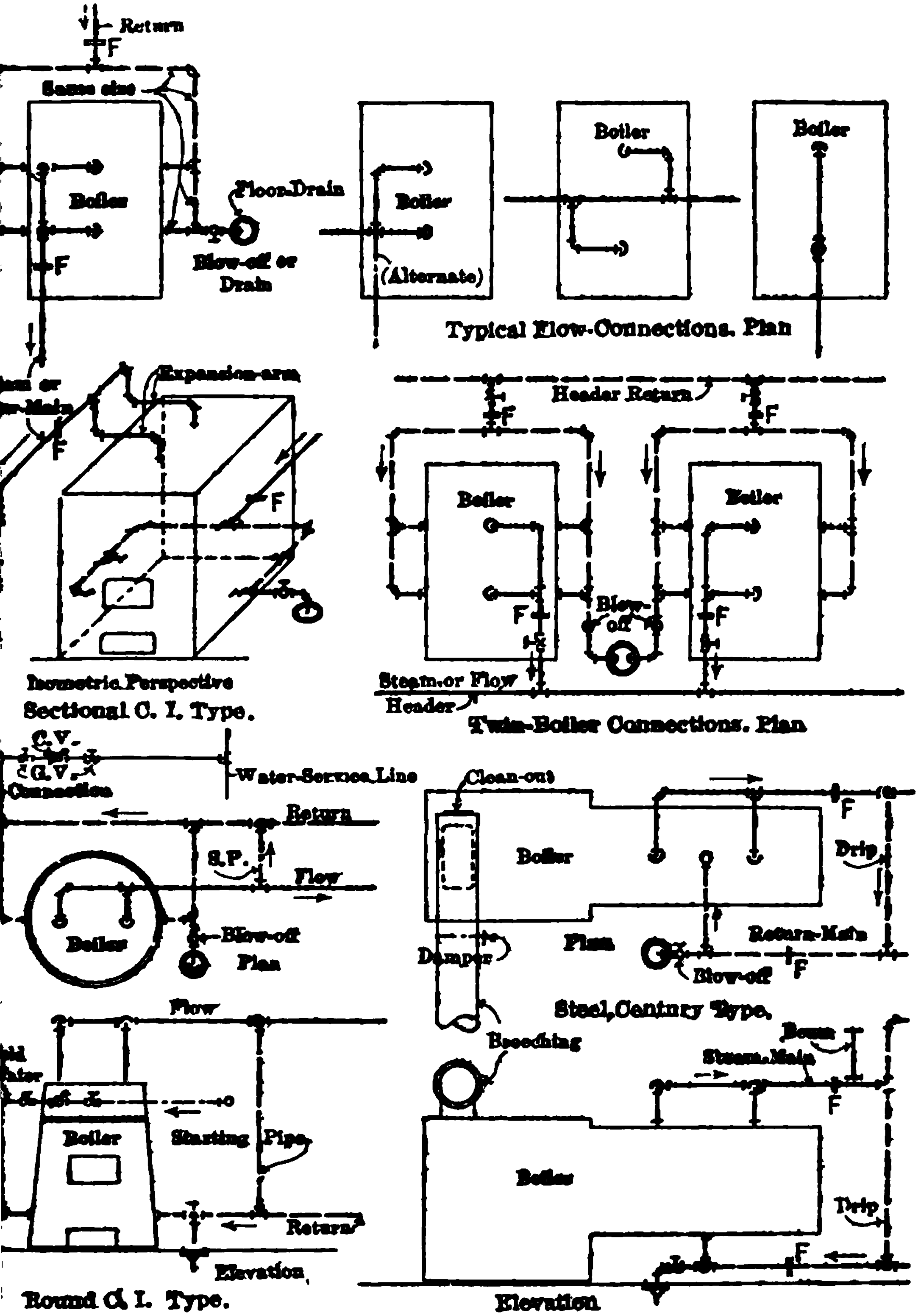


Fig. 73. Heating-boiler Connections

stable spindle (Fig. 70), which is supported centrally in the cast-iron floor-eve or flange, or else they have a non-adjustable spindle similarly supported, d are equipped with a control-damper. In either case the adjustable head or

damper must be locked positively in the finally adjusted position. In the c of concrete floors it is very desirable to use a cast-iron sleeve and flange (F 70) rather than a galvanized sleeve and cast-iron flange.

The Effect of Vitiated Air. The amount of CARBON DIOXIDE present vitiated air has been, until recently, quite generally understood to be the elem of danger that should be kept within safe limits. Dr. Ira Remsen has pointed that the presence of carbon dioxide in itself is not dangerous to health exc that it reduces the supply of oxygen by displacing it. Carbon dioxide is poisonous, but the ORGANIC IMPURITIES that are exhaled at the same time w other gases that are given off may prove a menace to health. The ill effects breathing air in a poorly ventilated room are due to the small quantities decomposing organic matter and unhealthful gases. The carbon dioxide g erated by the lungs and given off at the same time as the other impurities ser more or less as an indicator of the presence of the real danger. Any loweri of the oxygen-supply that is actually required for the proper and necess transformation of the potential heat-value of the food into the physical a nervous energy required to keep the human machine running, and to read supply the additional demand made upon that machine to perform exter work, means that industrial workers who perform their duties in a vitia atmosphere do so at the expense of a lowered vitality, and are naturally l productive. Satisfactory ventilation consists not only in constantly supplyi in a pure condition, fresh air FREE FROM DUST AND OTHER IMPURITIES, at t proper temperature and WITH THE PROPER AMOUNT OF MOISTURE PRESENT, b also in efficiently removing the vitiated air. This cannot be positively acco plished during the heating-season by simply opening the doors and windo Some mechanical means must be employed. Many physicians, however, not believe in MECHANICAL VENTILATION for hospitals, and advocate ventilat by the OPEN-WINDOW METHOD; and many hospitals are now constructed with any provision for mechanical ventilation except for the toilets and operati rooms for which exhaust-fans are provided.

Relation between Humidity and Temperature. The proper and health RELATIVE HUMIDITY OF THE AIR in buildings has only in recent years been gi the thought and attention it rightfully deserves. Heated or warmed whether purposely introduced into a building for warming, or naturally enter by infiltration, on being expanded by heat, has its percentage of moisture or re tive humidity lowered, and consequently its CAPACITY FOR ABSORBING MOIST greatly increased. There is, therefore, experienced the sensation due to so-ca DRY HEAT. This causes an excessive and unnatural evaporation of moist from the skin and from the membranes of the respiratory organs. Evaporat takes place by the direct application of heat and is essentially a refrigerating cooling process. The abstraction of heat from the body for this purpose, n urally tends to lower the surface-temperature, and one feels several degr cooler than the temperature recorded by the thermometer in the room. H. M. Smith's many observations and experiments upon the sensations produ by DIFFERENT PERCENTAGES OF SATURATION, led him to make the following st ment: "It may be accepted as a cardinal rule that if a room is at 68° and is warm enough for any healthy person, it is because the RELATIVE HUMIDIT too low." A STANDARD RELATIVE HUMIDITY may be obtained when mechan ventilation is used by the addition of a HUMIDIFIER to the system. The sub of AIR-CONDITIONING is fully treated in Heating and Ventilating, Vol. I, Harding and Willard.

Requirements for Good Ventilation. There is quite a diversity of opi among various authorities as to what constitutes GOOD VENTILATION in m

tances. The following data by G. D. Small represent good practice in this pect:

Types of Buildings. Air-Changes to be Allowed

FICE-BUILDINGS	Portions above grade	One change per hour.
	Basement, general	Four changes per hour.
	Mechanical plant	Ten changes per hour.

CTORY-BUILDINGS which have no mechanical or natural ventilation, one change per hour. For factories in which large doors from the outside are frequently opened, about four air-changes per hour.

SIDENCES which have loose windows, two changes per hour.

URCHES. Four changes per hour, except small rooms, which should have five or six changes per hour. These data for churches contemplate mechanical ventilation. The majority of public buildings and many of the factories require ventilation or the fan system of heating.

The Usual Requirements for Air Supplied per Person are as Follows

PITALS	Ordinary	from 35 to 40 cu ft per min
	Epidemic	80 cu ft per min

	Air-change
Detention-rooms	6 min
Toilet-rooms	6 min
Bath-rooms and duty-rooms	8 min
Kitchens	3 min
Serving-rooms	10 min
Fumigating-rooms	10 min

KSHOPS	25 cu ft per min
ONS	30 cu ft per min
ATERS	from 20 to 30 cu ft per min
TING-HALLS	20 cu ft per min
OLS	30 cu ft per min per child and 40 cu ft per min per adult

The Usual Time-Intervals for One Air-Change are as Follows

HOTELS

Room	Air-change	Room	Air-change
gine-room	6 min	Café	8 min
chen	1½–5 min	Lobby under balcony	8 min
staurant	6 min	Main lobby	20 min
e-toilet	5 min	Banquet-hall	15 min
iard	10 min	Retiring-room	10 min
ber-shop	8 min	Kitchens	8 min
ing-room	15 min	All others	15 min
m-room	12 min	Toilets	6 min
fet	8 min		

LIBRARIES

ridors	15 min	Inside rooms	8 min
ement-rooms	15 min	Corner rooms	7 min
ding-rooms	12 min	Toilet-rooms	5 min

ndries should have an air-change every 4 to 6 min.

E. Radiation on sides of buildings subjected to prevailing and cold winds should eased 10% up to the 10th floor and 15% above that floor.

Ventilation-Laws. The number of VENTILATION-LAWS has increased ve rapidly in the last few years, not only as regards the number of states whi have added such laws to their codes, but also as to the scope and effectiveness these statutes. In many cases a special ventilation-officer or commission h been appointed to see to the enforcement and extension of the requirements f compulsory ventilation, so that it behooves the architect or engineer to becom thoroughly familiar with the law of the state or states wherein he practic A summary of the law recently enacted by the legislature of the state of Oh is given in the following paragraphs as an example of the regulations wi which architects and engineers must conform in preparing plans and specific tions. This law as well as the law of Massachusetts, attempts to provide ve definite regulations for heating and ventilating all classes of buildings. Futu legislation in other states will undoubtedly take a more specific form, establishi complete and definite codes for the heating and ventilation not only of publ buildings but of workshops, factories and mercantile establishments as well.

Requirements of the Department of Inspection of the Industrial Commission of Ohio for the Heating and Ventilation of Public Buildings, Hospitals, Asylums and Homes

Temperature

A heating system shall be installed which will uniformly heat the vario parts of the building to the following temperatures in zero weather.

THEATERS AND ASSEMBLY-HALLS. All parts of the buildings, except storag rooms, 65° F.

CHURCHES. Auditorium, social and assembly-rooms, 65° F. All oth parts of the building, except storage-rooms, 70° F.

SCHOOL-BUILDINGS. Corridors, hallways, play-rooms, toilets, assembl rooms, gymnasiums and manual-training rooms, 65° F. All other parts of t buildings, 70° F.

HOSPITALS, ASYLUMS AND HOMES. Operating-rooms, 85° F. All other par of the buildings, except storage-rooms, 70° F.

Change of Air

The heating system shall be combined with a system of ventilation which normal temperature will change the air the following number of times, or supp to each person the following number of cubic feet of air per hour.

THEATERS. Parlors, retiring, toilet and check-rooms, six changes per hour

AUDITORIUMS. 1 200 cu ft of air per person per hour.

ASSEMBLY-HALLS. When used in connection with a school-building, lod building, club-house, hospital or hotel, six changes per hour; and in all oth assembly-halls, 1 200 cu ft of air per hour per person.

CHURCHES. Auditoriums, assembly-rooms and social rooms, six changes p hour.

SCHOOL-BUILDINGS. All parts of the buildings, except corridors, halls a storage-rooms, six changes per hour.

ASYLUMS, HOSPITALS AND HOMES. (1) Rooms with fixed capacity:

	Adults	Children	Babies
ospitals, contagious and epidemic...........	6 000	4 000	3 000
ospitals, surgical and medical...............	3 000	2 400	1 500
ɛnal institutions..........................	1 800	1 800	...
ll other buildings..........................	1 800	1 500	...

) Rooms with variable capacities:

Hospitals, contagious and epidemic.................. 12 times per hour
Hospitals, surgical and medical..................... 12 times per hour
All other buildings................................ 6 times per hour

Rooms accommodating four or less persons need not be provided with a system ventilation.

Radiators

No radiator shall be placed in any aisle, foyer or passageway of a new theater, embly-hall or church, but such radiators may be placed in recesses in the lls.

Registers

Io floor-registers shall be used in theaters, assembly-halls, or hospitals.
Io floor-registers, except foot-warmers, shall be used in a school-building.
loor-registers may be used in churches.
therwise all vent-registers shall be placed not more than 2 in above the floor-, and warm-air registers not less than 8 ft above the floor-line (except when registers are used when a change of air is not prescribed).

Systems to be Installed Where a Change of Air is Required

he system to be installed when a change of air is required shall be either a ity or mechanical furnace system, gravity indirect steam system, or hot-water ɛm; mechanical indirect steam or hot-water system, or split steam or hot-r system; except in hospitals, where a direct-indirect system may be used nnection with an exhaust-fan. The fresh-air supply shall be taken from de the building and no vitiated air shall be reheated. All vitiated air shall nducted through flues or ducts and be discharged above the roof of the ling.

CEPTIONS. Standard ventilating stoves may be used in the following ings:

sembly-halls seating less than 100 persons.
urches seating less than 100 persons.
school-buildings, hospitals, asylums and homes.

Furnaces

naces may be used in all classes of buildings.

Gravity Indirect Hot-Water or Steam-Radiator Systems

irect hot-water or steam-radiators shall be located in basement fresh-air directly at the base of masonry hot-air flues, and shall be properly con-l to same with galvanized-iron housing.

Indirect Radiating-Surface for Heating and Ventilating Purposes

One square foot of radiating-surface shall be provided to heat not more than the following number of cubic feet of air per hour:

Height	Steam	Hot water
First story	200	125
Second story	250	160
Third story	300	200
Fourth story	250	235

For Heating Wall-Surfaces and Glass-Surfaces. The amount of radiating-surface for the heating of the glass-surface and wall-surface shall not be less than that obtained by adding together the glass-surface and one fourth the exposed wall-surface, both in square feet, and multiplying by the following factors:

Height	Steam	Hot water
First story	0.7	1.05
Second story	0.6	0.9
Third story	0.5	0.75
Fourth story	0.4	0.5

Accelerating or Aspirating Coils for Vent-Flues. Vent-flues used in connection with a gravity indirect steam or hot-water system shall be provided with accelerating coils placed 1 ft above the vent-openings.

Mechanical Fan Plenum System

This system shall be designed with furnaces, tempering coils or blast-coils so as to furnish heated air, and is to have cleaning-screens, fan plenum chambers, galvanized-iron or masonry horizontal ducts, masonry hot-air flues, electric motor, gas or gasoline engine, or a low-pressure steam-engine operating on a steam-pressure not to exceed 35-lb gauge to operate fan and such other device as is necessary to make this a complete working system. All parts and apparatus in connection with the installation are to be of ample size to make a perfectly free and easily working system, which must thoroughly heat all portions of the building without forcing.

Velocity of Air

The velocity of the air traveling through ducts, flues, etc., shall never exceed the following number of feet per minute:

Ducts, Flues, etc.	Feet per minute
Fresh-air screens, small mesh	600
Fresh-air ducts, gravity system	300
Fresh-air ducts, mechanical system	850
Tempering coils, gravity system	300
Tempering coils, mechanical system	1 000
Furnaces, gravity system	400
Furnaces, mechanical system	900
Trunk-ducts, mechanical system	1 000
Laterals, branches and single ducts, mechanical system	750
Vertical flues, mechanical system	500

'ertical warm-air flues, gravity system, first story	300
'ertical warm-air flues, gravity system, second story	350
'ertical warm-air flues, gravity system, third story	390
ertical vent-flues less than 20 ft high	300
ertical vent-flues from 20 to 33 ft high	350
ertical vent-flues from 33 to 46 ft high	390
ertical vent-flues from 46 to 60 feet high	440
'arm-air registers	300
ent-registers	300

Maximum Speed of Fans

The maximum speed of fans used in connection with either an exhaust or num system of heating or ventilating, under normal conditions, shall never eed the following:

ameter of fan in inches	18	24	36	48	60	72	96	120	180
volutions per minute	700	550	400	300	225	175	150	125	75

Location of Heater-Room

No heater-room shall be located under the auditorium, stage, lobby, passage-, stairway or exit of a theater; nor, under any exit, passageway, public hall obby of an assembly-hall, church, school-building, asylum, hospital or home. s applies to new buildings, and a changed location of a heater-room in an ting building. No cast-iron boiler carrying more than 10-lb pressure or l boiler carrying more than 30-lb pressure shall be located within the main s of any school-building.

Standard Fire-Proof Heater-Room for New Buildings

ll furnaces and boilers, including the breeching, fuel-rooms and firing-spaces be enclosed by brick walls not less than 12 in thick, or by monolithic con- walls not less than 8 in thick. The ceiling over the same shall not be less the following: reinforced-concrete slab, 4 in thick; brick arches, 4 in thick, red with 1 in of cement mortar and supported by fire-proof steel with the ssary tie-rods; or hollow-tile arches, 6 in thick, covered with 2 in of concrete, ered on the under side and supported by fire-proof steel with the necessary ods.

Specifications for Furnace-Work

e following form is given as a guide to architects in preparing the specifi- ns for furnace-work:

FICATIONS FOR FURNACE-WORK IN RESIDENCE FOR MR..........TO BE AT..........................

..................ARCHITECT

nace. Furnish and set up complete, where shown on basement-plan, one ame . . .) furnace, or approved equal, portable-pattern, with double casings. ct the furnace with the chimney with a No. 22 galvanized-iron smoke-pipe same size as the collar on the furnace; all bends or turns to be made with piece elbows; the pipe to be strongly supported by wire, and to be kept below the ceiling.

Pit. Excavate for and build a cold-air chamber under the furnace not an 18 in deep, with 8-in brick walls, laid and plastered with cement; also

cement the bottom of the chamber. Build the cold-air duct under cellar-floor where shown on plan, — ft long, 14 in deep in the clear, and — in wide, with sides of hard brick in cement, and with the sides and bottom smoothly plastered with cement. Cover the duct with 3-in flagstones with tight joints, leaving opening of proper side for the wooden box to be built by the carpenter (wooden box should be included in carpenter's specifications).

Hot-Air Pipes. Furnish and properly connect with furnace and register-boxes leaders and stacks of the following sizes, all to be made of bright IX tin, and the stacks are to be double with an air-space. All turns in leaders to be made by three-piece or four-piece elbows, and the stacks to have boots or starters of approved pattern.

Sizes of Pipes and Registers

Hall	11" leader,	no stack	12" × 14" register
Parlor	11½" leader,	no stack	12" × 15" register
Dining-room	12" leader,	no stack	12" × 15" register
Library	10½" leader,	no stack	12" × 14" register
Chamber No. 1	10" leader,	4" × 15" stack,	10" × 14" register
Chamber No. 2	9" leader,	4" × 13" stack,	10" × 12" register
Chamber No. 3	8½" leader,	4" × 13" stack,	10" × 12" register

Registers. All registers are to be of sizes given in the foregoing list, of the (...name...) or approved equal, manufacture; japanned, except those in the first story, which are to be electro-bronze-plated. All floor-registers are to be set in iron borders corresponding with the registers.

Register-Boxes. All register-boxes to be made double; for first-story boxes the JOISTS ARE TO BE LINED WITH TIN and provided with CEILING-PLATES the full size of the registers, with plaster-collars attached, so that pipes and boxes can be removed without disturbing the plastering or defacing the ceiling.

Miscellaneous. All horizontal pipes in the basement are to be round, and where they pass through partitions they are to be provided with collars, so that the pipes can be removed without disturbing the plastering. All leaders are to be provided with dampers and tin tags designating the different rooms they supply; and whenever pipes run near woodwork the same is to be properly covered with tin and protected from any danger from fire. The contractor is to remove all rubbish made by him, clean up all ironwork, leave the whole apparatus in complete working order, and furnish a poker of proper size.

Guarantee. The contractor is to guarantee, if he furnishes the heating drawings, that the furnace shall, under proper management, heat all rooms with registers connected with the furnace, to 70° F., when the temperature outside indicates 0°. In the event of the failure of the furnace to do this the contractor, at his own expense and without unnecessary delay, is either to make the furnace heat said rooms or substitute another furnace that will heat them.

Hot-Air-and-Water Combination-Furnaces

Combination-Furnaces. It is quite difficult, if not impossible, to heat dwellings covering throughout, more than 1 400 sq ft with warm air alone. On account of the much larger exposure and the increased length of leaders it becomes necessary to supplement the warm air with an auxiliary heat whi

be carried to remote and exposed parts of the house, and which will not ffected by pressure of wind or long and crooked pipes. For supplying this iary heat, hot water has been found best adapted as a rule, and a variety OMBINATION-FURNACES are now made which contain provisions for heating r which may be carried by pipes to radiators located in those parts of the e most difficult to heat by warm air. Such combination-systems have used with success. The construction of the parts for heating the water s with different makes of furnaces. Some furnaces have a portion of the ot hollow, and the water is heated there; others have a separate heater nded over the fire-pot. As a rule, the parts of the house which should eated by the hot water are the halls, bath-rooms, and perhaps the s on the north or west sides of the house. The same rules govern the of the radiators and piping and the manner of installing as in an entire ater plant.

Specification for Hot-Water Heating-Apparatus in a Residence

s specification contemplates a complete upfeed two-pipe gravity hot-water g system, to be installed in accordance with the drawings covering the

ter. Furnish and set up in cellar, where shown on plan, one (. . . name . . .) boiler, or approved equal, guaranteed free from all flaws and defects. eater to be set on a substantial foundation of hard brick laid in cement r and put in by the heating-contractor. Furnish and deliver one set of ols, consisting of one poker, one slice-bar and one fine brush and handle.

ke-Pipe. Connect the boiler to the chimney by means of smoke-pipe of No. 22 galvanized iron, the diameter of the pipe to be equal to the on the heater.

mings. The boiler is to be provided with one thermometer registering o° F. to 250° F., and one Standard altitude gauge.*

er-Connections and Blow-off. Feed-water with its supply-pipe will be t within 6 ft of the boiler by the plumber and left with one ¾-in cast-iron for boiler-connection, which is to be made by this contractor, with suit-ck. Draw-off cock to be placed on lowest point of system and to be or hose-attachment.

. Furnish and run all necessary flow and return-mains of ample size, ing them to radiators with risers of ample size to insure the free flow vater to and from the radiators. All connections from risers to radiators ade below floors.

ity of Materials. All materials used in the construction of this apparatus e the best of their respective kinds, all fittings to be heavily beaded and f the best gray iron with clean-cut threads, and, when practicable, Y's L's are to be used.

ing. The ends of all pipes used in the construction of this apparatus e reamed and all obstructions removed before pipes are placed in position. and return-mains in the basement are to be supported by neat, strong, le hangers, arranged to suit expansion and contraction, and properly to timbers overhead. At all points where pipes pass through ceilings,

* ltitude-gauge indicates the amount of water in the system and is a convenient at which avoids the necessity of consulting the gauge-glass in the tank. It pensed with if desired.

floors, or partitions, tin thimbles are to be provided and the holes protect with floor or ceiling-plates.

Expansion-Tank. The expansion-tank is to be constructed of galvanized ir and is to be furnished with a proper gauge-glass with brass mountings comple It is to be placed at least 3 ft above the highest radiator in a suitable pl and supported on a proper shelf. From this tank an overflow-pipe will run to the basement or other suitable place with a vent-pipe through the r properly flashed.

Radiators. Furnish, set up, and pipe the following radiators:

Rooms	Number of radiators	Radiating-surface, sq ft
Main hall	1 indirect radiator	108
Sitting-roon	1 indirect radiator	120
Library	1 direct radiator	40
Dining-room	1 direct radiator	60
Sitting-room chamber	1 direct radiator	40
Library chamber	1 direct radiator	44
Dining-room chamber	1 direct radiator	36
Kitchen chamber	1 direct radiator	32
Bath-room	1 direct radiator	32
	9 radiators	512

In all there are to be 284 sq ft of direct surface and 228 sq ft of indirect; to surface, 512 sq ft. The direct radiators to be (...name...) hot-water patte or approved equal, 38 in high.

Air-Valves. Each radiator is to be provided with a nickel-plated key-ty air-valve.

Radiator-Valves. Each direct radiator is to be promptly connected to t system of piping with a quick-opening nickel-plated radiator-valve and un elbow.

Indirect Radiation. The indirect radiators are to consist of two stacks (...name...) hot-water radiation, or approved equal, connected toget with tight joints and firmly suspended from the basement-ceiling by suita wrought-iron hangers. The stacks are to be so piped and hung as to per a noiseless and constant flow throughout of the heated water. Each st is to be enclosed in a galvanized-iron chamber with proper fresh-air inlet-d and a corresponding outlet-duct for warm air, connected to the register in t room which the stack is intended to heat. The registers are to be of t (...name...) pattern, electro-bronze-plated, and of the following sizes: h 12 by 19; sitting-room, 14 by 22 in. Registers are to have floor-borders a to be set in register-boxes. The duct connecting the stack and register be so arranged that all fresh air coming in will be properly heated and c veyed, with least loss, to its destination. In arranging indirect boxes, c is to be exercised in getting ample space for cold air under the stack, an corresponding space for warm air over the stack.

Covering of Pipe. All flow and return-pipes and fittings in cellar above floor are to be properly covered with 1-in hair-felt neatly sewed up in can

nd painted one coat of good white lead, or covered with asbestos or magnesia ctional covering, with canvas cover, and secured by lacquered-brass bands.

Boiler-Covering. All exposed parts of the boiler, except the front, are to e covered with plastic asbestos, 1½ in thick, neatly applied and troweled, nooth.

Workmanship. All work is to be done in a neat, substantial and workman- e manner, and the apparatus, when completed, is to be thoroughly tested d left in good working order.

Guarantee. The contractor is to guarantee, if he is to furnish the heating- wings, that the apparatus he installs will be of ample capacity to evenly intain a temperature of 70° F. in the rooms in which radiators are located. en the outside temperature is at zero, and that the apparatus throughout l have a free circulation when in operation.

Steam-Heating for Residences

General Requirements. For very large residences, the author would ommend steam-heat, all of the principal rooms to be heated by indirect iation, and only the bath-room, halls, and perhaps the attic and one or two ns on the north side, which generally includes the dining-room, by direct ation. For dining-rooms a special direct radiator, containing a warming- et, is made. The air-supply to the indirect stacks should be very large provided with a damper, so that the supply may be regulated according he weather. The boilers used in residence-heating are generally of the cast- sectional type described on page 1278. The single-pipe system is com- ly used in dwellings, all indirect radiators, however, being two-pipe.

Specification for a Low-Pressure Steam-Heating Apparatus for Heating by Direct Radiation

tention. This specification is intended to cover everything necessary to finish and install in the above-mentioned building a complete steam-heat- ystem in strict accordance with the plans and this specification, as prepared ———, architect.

ns. The drawings herewith are intended to show only the location of the r, piping and radiators; the arrangement of the piping will be left largely e contractor, subject to the approval of the architect.

neral Requirements. This contractor is to provide all necessary tools and nces for the erection and completion of the work, and when completed, remove all apparatus, refuse and débris from the building and grounds, g the work in a clean, uninjured and perfect condition. No cutting of escription tending to weaken the building structurally is to be undertaken ut consulting the architect. This contractor is to be fully responsible for fety and good condition of the work and material embraced in this contract he completion and acceptance of the same. All work is to be of the best y, and should at any time improper, imperfect, or unsound material or workmanship be observed, whether before or after same has been built e structure, this contractor, upon notice from the architect, is to remove nd substitute good and proper material and workmanship without delay e thereof, in default of which the architect is to effect same by other as may be deemed best, and is to deduct the cost of such alterations from n due the contractor under this contract.

System. The heating is to be effected by direct radiation distributed throu out as shown on the drawings, and the circulation of the steam is to be by one-pipe circuit system.

Boiler. This contractor is to build the foundation for the boiler, whi shown, 12 in deep, of common hard brick laid in cement mortar. He is leave an ash-pit for the boiler of proper size, 12 in deep, cemented, and ma water-tight. He is to furnish and set up one (...name...) cast-iron section or approved equal, boiler, provided with 6-in low-pressure brass-cased stea gauge, water-gauge, and glass, gauge-cocks, combination-column, safety-valv and blow-off valves, and all other usual and necessary trimmings to compl the boiler,* and a full set of fire-tools, consisting of one slicing-bar, one h one poker, and a cleaning-brush. He is to cover the boiler with 1 ½-in of asbes cement, neatly troweled to a smooth finish.

Water-Supply. The plumber is to bring the water-supply to within 6 ft boiler, but this contractor is to make connection with boiler with ¾-in ir pipe, stop-cock and check-valve.

Smoke-Pipe. Contractor is to connect the boiler with the chimney with round smoke-pipe made of No. 22 galvanized iron with suitable balance-dampe This connection to be of same size as left for this purpose by maker of boiler.

Main Pipes and Risers. The steam-main is to be run full size for the enti length and provided with an automatic air-vent at the end of the run. It to be of ample size to carry all the risers and radiators attached to the syste and is to be graded 1 in in 10 ft in the direction of the flow. From the top this main the various branches are to be taken to radiators and risers, t connections for which are to be so made that no traps are formed. If a tr cannot be avoided, a drip connected with the return-main is to be installe Radiators on first story are to be connected direct to steam-main. Radiato for the second and third floors may be taken off the same riser. The mai after serving the last radiator, is to drop below the water-line of the boil and its size reduced, and it is to run back to the boiler as a wet-return-mai The steam-main at the end of the run is to be 24 in or more above the water-li of the boiler. The boiler is to be installed in a pit if necessary to accompli this.

Pipes and Fittings. All pipe used throughout is to be of the best quali wrought-iron or steel pipe of standard weight and thickness, with the en reamed, free from imperfections, and true to shape. All threads are to clean-cut, straight and true. All fittings are to be of the best heavy gray ir with taper-threads, and are to be heavily beaded. No inferior pipe or fitti will be allowed.

Supports. All piping is to be supported by approved expansion-hangers rollers, not to exceed 10 ft apart. Neat cast-iron floor and ceiling-plates to be used where pipes pass through floors, ceilings and partitions.

Radiators. Direct radiation is to be furnished to the amount enumerated the drawings of the (...name...) make, or approved equal.

Radiator-Valves. The radiators are to be furnished with removable di type union valves, rough nickel-plated, and are to have hard-wood hand-whee

Air-Valves. Radiators throughout the entire building are to be furnis with (...name...) automatic air-valves, or approved equal.

* For house-heating plants it is well to specify also "one automatic damper-regul of approved pattern, with connection for operating draft-door and cold-air check."

Pipe-Covering. All pipes in the cellar above the floor are to be covered th 1-in asbestos (or magnesia) sectional covering with canvas cover and ured by lacquered-brass bands.

Painting and Bronzing. All radiators and exposed pipes in rooms or halls to be neatly painted two coats of best radiator-enamel, or bronzed in desired ors.

Finally. When completed, the apparatus is to be tested to 10-lb steam-ssure and made tight at that pressure, said test to be conducted under the ervision of the architect. Fuel for the test is to be furnished by the owner, when accepted, the apparatus is to be turned over to the owner in com-e working order. All valves and stuffing-boxes are to be properly packed the plant completed in all its parts, it being understood that this contractor furnish all miscellaneous material, tools, labor, etc., necessary to complete work in a first-class and workmanlike manner.

uarantee. This contractor is to guarantee that when the apparatus is pleted it will be free from all mechanical defects and, if he is to furnish design and layout, that the installation shall be of ample capacity to heat ooms where radiation is placed to a temperature of 70° F. when the outside perature is 0° F.

CHIMNEYS*

By

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Draft. To burn a fuel at a given rate (pounds per square foot of grate-surfa[ce] per hour) requires a definite weight of air to be supplied for combustion. Th[e] air passes under the grate and through the fuel-bed and meets with considerab[le] resistance in its flow, not only through the fuel-bed, but through or around th[e] boiler-tubes and smoke-flue or breeching. The motive force causing the air-flo[w] in a natural-draft plant is supplied by the chimney. The difference betwe[en] the atmospheric pressure and the pressure existing at any point in the furna[ce] or in the flue is termed the DRAFT at that particular point. This pressure [is] ordinarily measured by means of a U tube filled with water, the draft bein[g] recorded in inches of water, and is the difference in the heights of the wate[r] columns in the two legs of the U tube.

Height. The INTENSITY OF DRAFT that a chimney is capable of producing [at] the base is a function of its height, the temperature of the flue-gases, and th[e] temperature of the outside air, which is generally assumed to be 60°. The tem[-]perature of the flue-gas is ordinarily assumed to be 550°. The intensity [of] draft produced, per foot height, measured in inches of water is

$$H = 0.0071\, L$$

L = height of chimney above grate, in feet. The flue-gas temperature is tak[en] at 550° and the outside temperature at 60°. Ordinarily $0.8H$ is taken as rep[-]resenting the AVAILABLE DRAFT, in order to allow for the cooling of the chimne[y] gases. Then $0.8H$ must be equal to or greater than the sum of the expecte[d] draft-losses as given in the following paragraphs.

Draft-Losses. The DRAFT-LOSSES through the fuel-bed depends upon th[e] rate of combustion required and the kind of fuel. This loss may be approx[i-]mated by using the data in Table I.

The LOSS OF DRAFT between the grate or furnace and a point just beyond th[e] damper-box of a boiler is about as shown in Table II when the boilers are oper[-]ated at normal rating; bituminous coal burned at the rate of from 25 to [3x] lb per sq ft of grate-surface per hour.

The loss of draft through the boiler will depend largely upon the method [of] baffling employed, and increases with the per-cent rating at which the boiler [is] operated. The precipitating-figures should be increased by approximate[ly] 55% when the boiler is operated at 150% of its rated capacity, and by 75[%] when it is run at 200% rating.

Velocity of Gases through Flue and Chimney. In preliminary estimat[es] 5 lb coal per boiler horse-power developed, and 24 lb air per lb of coal is usual[ly]

* See, also, Chimneys for Heating Boilers, page 1281; Flues for Kitchen Ranges [and] Fireplaces, page 1282, and Selection of Chimney Flues, page 1282.

Table I. Loss of Draft between Furnace and Ash-Pit to Burn Coal

Kind of coal	Combustion-rate, *R*, in pounds of dry coal per square foot of grate per hour						
	15	20	25	30	35	40	45
	Force of draft in inches of water						
, Ind., Kan., bituminous.....	.14	.20	.26	.33	.40	.48	.57
., Ky., Pa., Tenn., bituminous	.16	.23	.31	.40	.49	.60	.72
., Pa., Va., W. Va., semibitu-inous..................	.18	.26	.35	.45	.57	.71	.87
hracite pea................	.30	.45	.64	.88	1.23	...	...
hracite buckwheat No. 1....	.43	.68	1.00	1.50	...	...	...

II. Loss of Draft between Grate or Furnace and a Point Just beyond Damper-Box

Horizontal return tubular..	.25 to .30 in of water
Babcock & Wilcox.........	.20 to .35 in of water
Stirling..................	.51 in of water
Vertical tubular...........	.43 in of water

ed. The customary allowable VELOCITIES OF GASES in chimneys, when sign is based on 120 lb of the flue-gas per hour per rated boiler horse-power, from 17 ft per sec for a diameter of stack equal to 24 in, to 31 ft per sec 72-in or larger diameter. These figures correspond to a weight of 0.68 10 lb per sq ft of area. The formula that is supposed to give the most nical diameter for an unlined steel chimney or stack, and used by many ers in this country is $d = 4.68\sqrt[5]{(\text{h.p.})^2}$, in which d is the inside diameter es and *h.p.* is the rated capacity of the boilers served.

following figures are frequently used by engineers for approximating the draft in flues or breechings:

Horizontal flues, square or rectangular, from 0.13 to 0.15 in of water per Increase these values 50% for brick-lined flues. Loss of draft for easy ngle bends, 0.05 in of water.

When economizers are to be installed the temperature of the flue-gas is l to from 250° to 325°, and the total head, H, should be calculated on a these temperatures.

The loss of draft through the economizers should not be figured less 3 in of water.

The turns which the flue makes in leaving the damper-box of the boiler, enters the main flue and at the stack, should be considered and allowed

t is customary to make the flue or breeching approximately from 10 to eater in area than the stack to which it connects. The cross-section is re-n proportion to the volume of gas to be handled as the flue passes the n succession. The width of the flue or breeching, where it enters the

chimney, should never exceed one third the outside diameter of the chimney its base.

Example. The method of procedure in determining the dimensions o chimney and breeching is explained in the following example.

Three 150-1 h.p. return tubular boilers with a total of 1 500 sq ft of heati surface are to be served. The total area of the grate-surface is 90 sq ft. measured length of the breeching is 40 ft. The gas makes two right-angle tu one at the entrance to the breeching, and one on entering the chimney. fuel assumed is Pennsylvania bituminous coal. If 5 lb of coal per boiler ho power per hour is assumed as the fuel-consumption, the rate of combustio $(3 \times 150 \times 5)/90 = 25$ lb per sq ft of grate-surface per hour.

The weight of flue-gas per second is

$$(3 \times 120 \times 150)/(60 \times 60) = 15 \text{ lb}$$

Assuming a temperature of 550°, the volume of the flue-gas per secon $15/0.0393 = 382$ cu ft. Assuming an allowable velocity through the chimn area of 25 ft per sec, the required area is,

$$382/25 = 15.3 \text{ sq ft}$$

corresponding to 54-in diam, approximately The area of the flue is to 15% greater, or

$$15.3 \times 1.15 = 17.6 \text{ sq ft}$$

at the last boiler next to the chimney. The chimney must produce suffici draft to overcome the following resistance. The loss of draft through fuel- based on a rate of combustion of 25 lb per sq ft per hr (Table I) is 0.31 in. loss of draft through return tubular boilers (Table II) is 0.27 in. The los draft through the breeching is

$$0.15 \times 40/100 = 0.06 \text{ in}$$

The loss of draft occasioned by two turns is

$$2 \times 0.05 = 0.10 \text{ in}$$

The total loss is

$$0.31 + 0.27 + 0.06 + 0.10 = 0.74 \text{ in}$$

Then

$$H = 0.74/0.8 = 0.92 \text{ in}$$

or approximately 1 in.

Substituting this value of H in the equation

$$H = 0.0071L$$

the height, L, of the stack is

$$1/.0071 = 140 \text{ ft}$$

measured above the grate.

Kent's Chimney-Formulas. The following chimney-formulas by Wi Kent are largely used by engineers in this country: The formula is base the assumption that the friction-head in the chimney is considered equiv to a diminution of the area by an amount equal to a lining of inert gas, 2 thickness.

Table III. Size of Chimneys for Steam-Boilers

Kent's Formula

Diameter,	Area,	Effective area, $E = A - 0.6\sqrt{A}$	Height of chimney in feet														Equivalent square chimney. Side of square, $\sqrt{E}+4$
			50	60	70	80	90	100	110	125	150	175	200	225	250	300	
in	sq ft	sq ft	Commercial horse-power of boiler *														in
18	1.77	.97	23	25	27	29	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	16
21	2.41	1.47	35	38	41	44	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	19
24	3.14	2.08	49	54	58	62	66	.....	.....	.....	.....	.....	.....	.....	.....	.....	22
27	3.98	2.78	65	72	78	83	88	.....	.....	.....	.....	.....	.....	.....	.....	.....	24
30	4.91	3.58	84	92	100	107	113	119	.....	.....	.....	.....	.....	.....	.....	.....	27
33	5.94	4.48	.....	115	125	133	141	149	156	.....	.....	.....	.....	.....	.....	.....	30
36	7.07	5.47	.....	141	152	163	173	182	191	204	.....	.....	.....	.....	.....	.....	32
39	8.30	6.57	.....	.....	183	196	208	219	229	245	.....	.....	.....	.....	.....	.....	35
42	9.62	7.76	.....	.....	216	231	245	258	271	289	316	.....	.....	.....	.....	.....	38
48	12.57	10.44	.....	.....	.....	311	330	348	365	389	426	.....	.....	.....	.....	.....	43
54	15.90	13.51	.....	.....	.....	.....	427	449	472	503	551	595	.....	.....	.....	.....	48
60	19.64	16.98	.....	.....	.....	.....	536	565	593	632	692	748	.....	.....	.....	.....	54
66	23.76	20.83	.....	.....	.....	.....	.....	694	728	776	849	918	981	.....	.....	.....	59
72	28.27	25.08	.....	.....	.....	.....	.....	835	876	934	1 023	1 105	1 181	1 253	.....	.....	64
78	33.18	29.73	.....	.....	.....	.....	.....	.....	1 038	1 107	1 212	1 310	1 400	1 485	1 565	.....	70
84	28.48	34.76	.....	.....	.....	.....	.....	.....	1 214	1 294	1 418	1 531	1 637	1 736	1 803	2 005	75
90	44.18	40.19	.....	.....	.....	.....	.....	.....	.....	1 496	1 639	1 770	1 893	2 008	2 116	2 318	80
96	50.27	46.01	.....	.....	.....	.....	.....	.....	.....	1 712	1 876	2 027	2 167	2 298	2 423	2 654	86
102	56.75	52.23	.....	.....	.....	.....	.....	.....	.....	1 944	2 130	2 300	2 459	2 609	2 750	3 012	91
108	63.62	58.83	.....	.....	.....	.....	.....	.....	.....	2 090	2 399	2 592	2 771	2 939	3 098	3 393	96
114	70.88	65.83	.....	.....	.....	.....	.....	.....	.....	.....	2 685	2 900	3 100	3 288	3 466	3 797	101
120	78.54	73.22	.....	.....	.....	.....	.....	.....	.....	.....	2 986	3 225	3 448	3 657	3 855	4 223	107
132	95.03	89.18	.....	.....	.....	.....	.....	.....	.....	.....	3 637	3 929	4 200	4 455	4 696	5 144	117
144	113.10	106.72	.....	.....	.....	.....	.....	.....	.....	.....	4 352	4 701	5 026	5 331	5 618	6 155	128

If A = the actual area in square feet;
E = the effective area in square feet;
D = the diameter in feet;

Then $E = A - 0.60\sqrt{A}$.

The draft-power of a chimney varies directly as the effective area, E, and as the square root of the height, L. The formula for the horse-power of a chimney will take the form, $h.p. = CE\sqrt{L}$, in which C is a constant. The value of C as obtained by Kent from an examination of a large number of chimneys is 3.33 when 5 lb of coal is burned per boiler horse-power per hour.

The formula for the horse-power rating of a chimney is, therefore,

$$\text{h.p.} = 3.33E\sqrt{L} = 3.33\,(A - 0.6\sqrt{A})\sqrt{L}$$

or

$$E = 0.3\ \text{h.p.}/\sqrt{L}$$

The Babcock & Wilcox Company recommend that when the fuel used is low grade bituminous coal of the Middle or Western States, the sizes given in Table III be increased from 25 to 60%, depending upon the nature of the coal and the capacity desired. If the gas makes more than two turns it is advisable to increase the diameter given in the table by one size. The height must be increased at least 30% if economizers are used. Table III may be applied to heating-boilers, the equivalent rating in square feet of direct radiation being approximately equal to the horse-power rating × 100.

Chimneys for Tall Office and Loft-Buildings. The chimney or stack for a tall building is a special case in which the height is frequently fixed by the height of the structure itself or the height of the adjoining buildings. In this case a diameter is assumed and the method outlined in the preceding example applied.

General Formulas for the Design of Brick Chimneys. See Fig. 1

Let P = horizontal wind-pressure in pounds per square foot, ordinarily assumed as 25 lb per sq ft for round chimneys

xx = any section distant z from top of chimney

$z\left(\dfrac{d + d_1}{2}\right)$ = projected area above xx

R = horizontal wind-load in pounds

$= Pz\left(\dfrac{d + d_1}{2}\right)$

y = distance from xx to center of gravity of portion above xx

M = wind-moment in foot-pounds

$= Pzy\left(\dfrac{d + d_1}{2}\right)$

Properties of Section

d_1 = outside diameter

d_2 = inside diameter

$c = d_1/2$

I = moment of inertia of section

A = area of section in square feet

$= 0.7854\,(d_1^2 - d_2^2)$

$\dfrac{I}{c}$ = section-modulus

$\frac{I}{c} = \frac{0.0982\,(d_1^4 - d_2^4)}{d_1}$

W = weight of chimney above xx, in tons

S_1 = compressive stress at edge on leeward side due to W, in tons per square foot

S_2 = compressive stress at edge on leeward side due to M, in tons per square foot

$S_1 = +\frac{W}{A} \qquad S_2 = \pm \frac{Mc}{I}$

$$\text{Windward side, } S_w = \frac{W}{A} - \left[\frac{Mc}{I}\text{ (tension)}\right]$$

$$\text{Leeward side, } S_l = \frac{W}{A} + \left[\frac{Mc}{I}\text{ (compression)}\right]$$

S_w and S_l should not exceed the following values, in tons per square foot, for dial Brick Chimneys:

Maximum tension		Maximum compression	
Below 150 ft........	2 to 2½	200 ft and below.........	19
From 150 to 200 ft...	1 to 1½	Above 200 ft.............	21
Above 200 ft........	0		

'oundations. Calculate wind-moment, M_1 for chimney above ground-line.

$M_1 = Ph\,y_1 \left(\frac{d + d_2}{2}\right)$

l = length of side of square base in feet

$l_1 = l^2$ = area of base in square feet

$\frac{I}{c} = \frac{l^3}{6}$ = section-modulus of base

W_1 = combined weight of chimney and foundation

ample. It is required to determine the maximum compression, in tons per t at the base of the column, for the chimney shown in Fig. 2, and also the imum soil-pressure in tons per sq ft. The assumed wind-pressure is 25 lb q ft. (See General Formulas for the Design of Brick Chimneys, and Fig. 1.)
he area of section at base, $A = 0.7854\,(16^2 - 12.3^2) = 80.9$ sq ft.
he section-modulus at base, $I/c = [0.0982(16^4 - 12.3^4)]/16 = 257$.
he total weight of brick column (Table V) is $W = 495$ tons (interp olated).
he projected area of column is $\frac{1}{2} \times (8.75 + 16) \times 180 = 2\,228$ sqft.
he horizontal wind-load, $R = 2\,228 \times 25 = 55\,700$ lb = 27.8 tons.
he moment-arm of R is $y = \frac{1}{3} \times 180[(2 \times 8.75) + 16]/(8.75 + 16) = 81$ ft.
he wind-moment, $M = 81 \times 27.8 = 2\,252$ ft tons.
$= 495/80.9 = 6.2$ tons per sq ft.
$= \pm 2\,252/257 = 8.7$ tons per sq ft.
e maximum compression on the leeward side, $S_1 + S_2 = 6.2 + 8.7 = 14.9$ per sq ft. The maximum tension on the windward side, $S_1 - S_2 = -2.2$ per sq ft. The following computations are for a square base:
undation. The length of base, $l = 25.5$ ft, $A_1 = l^2 = 650$ sq ft, $I/c =$ /6 = 814. The weight of foundation, based on 1.9 tons per cu yd, is 266 The weight of the 4½-in lining is $36 \times 11 \times 0.063 = 25$ tons. The total t of column, lining and foundation, is $W_1 = 495 + 25 + 266 = 786$ tons. e moment-arm for R may be assumed the same as before, or 81 ft. Ther 2 252 ft-tons. The section-modulus of the base, $I/c = l^3/6 = 814$.
$= 786/650 = 1.2$ tons per sq ft. $S_2 = 2\,252/814 = 2.8$ tons per sq ft.
e maximum soil-pressure, $S_1 + S_2 = 1.2 + 2.8 = 4$ tons per sq ft.

$+S_1$

$+S_2=$

$+S_3=$

Tons per square foot

$$S_1 = \frac{W_1}{l^2} = \text{compression per sq ft due to } W_1$$

$$S_2 = \frac{6M_1}{l^3} = \text{compression per sq ft due to } M_1$$

Fig. 1. Details of Construction of Tall Brick Chimney

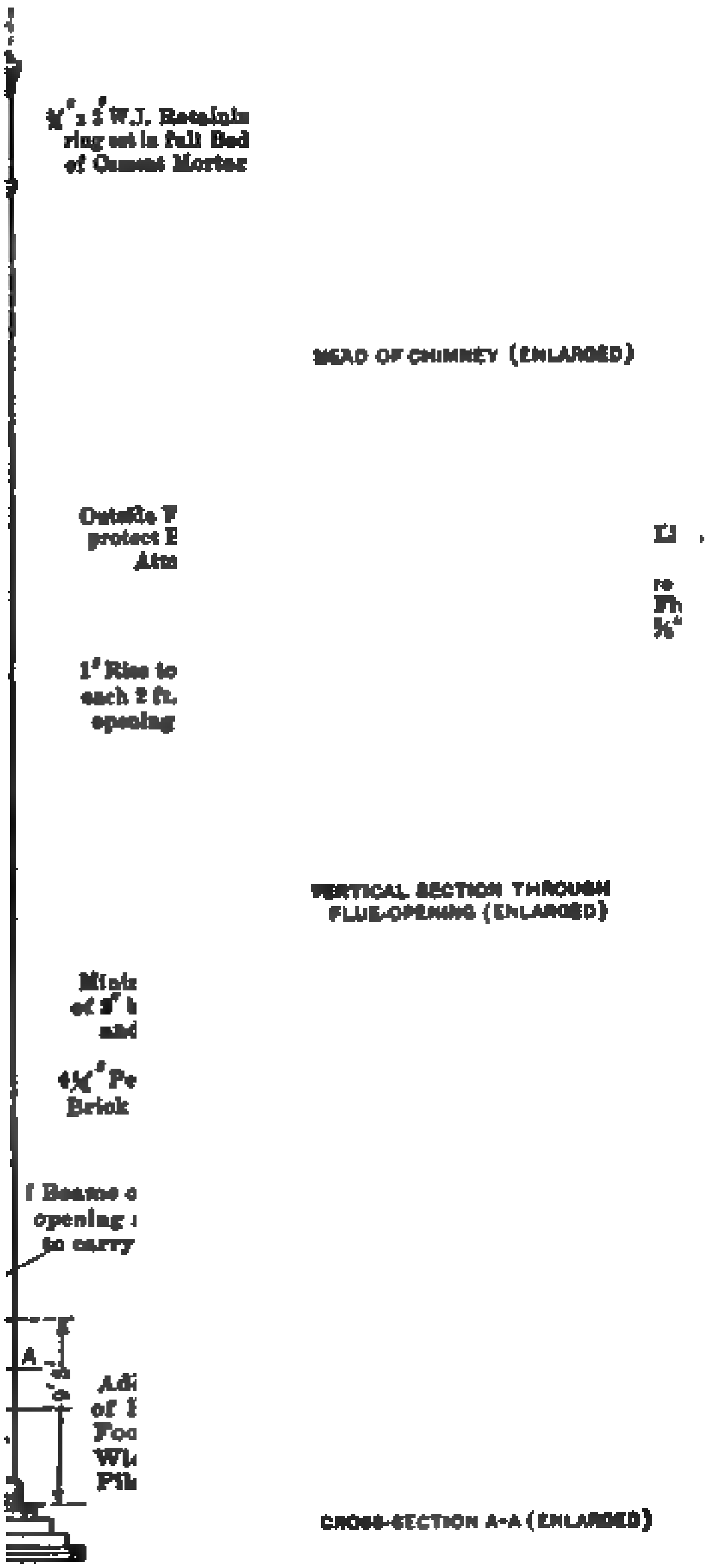


Fig. 2. Details of Tall Radial-brick Chimney

Height of chimney in feet	Internal diameter at top														
	3′	3′6″	4′	4′6″	5′	5′6″	6′	6′6″	7′	7′6″	8′	8′6″	9′	9′6″	10′
	Diameters at bottom of column, in feet														
75	7.42	7.69	7.96	8.46	8.96	9.46	9.96	......	......	......	......	......	......	......	......
80	7.80	8.04	8.27	8.70	9.13	9.58	10.02	......	......	......	......	......	......	......	......
85	8.18	8.38	8.58	8.95	9.31	9.70	10.08	......	......	......	......	......	......	......	......
90	8.57	8.73	8.88	9.18	9.48	9.81	10.13	......	......	......	......	......	......	......	......
95	8.95	9.07	9.19	9.43	9.66	9.93	10.19	......	......	......	......	......	......	......	......
100	9.33	9.42	9.50	9.67	9.83	10.04	10.25	10.75	11.25	11.75	12.25	......	......	......	......
105	9.70	9.78	9.85	10.03	10.21	10.38	10.55	11.03	11.50	11.95	12.40	......	......	......	......
110	10.06	10.13	10.20	10.40	10.60	10.73	10.85	11.30	11.75	12.15	12.55	......	......	......	......
115	10.43	10.49	10.55	10.77	10.98	11.07	11.15	11.58	12.00	12.35	12.70	......	......	......	......
120	10.79	10.85	10.90	11.14	11.37	11.41	11.45	11.85	12.25	12.55	12.85	......	......	......	......
125	11.16	11.21	11.25	11.50	11.75	11.75	11.75	12.13	12.50	12.75	13.00	13.50	14.00	14.50	15.00
130	......	......	11.65	11.88	12.10	12.12	12.13	12.47	12.80	13.09	13.37	13.80	14.22	14.69	15.15
135	......	......	12.05	12.25	12.45	12.48	12.51	12.81	13.10	13.42	13.73	14.08	14.43	14.87	15.30
140	......	......	12.45	12.63	12.80	12.85	12.90	13.15	13.40	13.75	14.10	14.38	14.65	15.05	15.45
145	......	......	12.85	13.00	13.15	13.22	13.26	13.49	13.70	14.08	14.46	14.66	14.86	15.23	15.60
150	......	......	13.25	13.38	13.50	13.58	13.66	13.83	14.00	14.42	14.83	14.96	15.08	15.42	15.75
155	......	......	13.58	13.73	13.87	13.97	14.06	14.18	14.30	14.68	15.06	15.19	15.31	15.61	15.91
160	......	......	13.92	14.08	14.23	14.35	14.46	14.53	14.60	14.95	15.30	15.43	15.55	15.81	16.07
165	......	......	14.25	14.43	14.60	14.73	14.86	14.88	14.90	15.22	15.53	15.66	15.78	16.00	16.22
170	......	......	14.59	14.78	14.96	15.11	15.26	15.23	15.20	15.49	15.77	15.90	16.02	16.20	16.38
175	......	......	14.92	15.13	15.33	15.50	15.66	15.58	15.50	15.75	16.00	16.13	16.25	16.40	16.54
180	......	......	......	......	......	......	......	......	15.80	16.05	16.30	16.40	16.50	16.65	16.80
185	......	......	......	......	......	......	......	......	16.10	16.35	16.60	16.68	16.75	16.91	17.06
190	......	......	......	......	......	......	......	......	16.40	16.65	16.90	16.95	17.00	17.16	17.31
195	......	......	......	......	......	......	......	......	16.70	16.95	17.20	17.23	17.25	17.41	17.57
200	......	......	......	......	......	......	......	......	17.00	17.25	17.50	17.50	17.50	17.67	17.83
205	......	......	......	......	......	......	......	......	......	......	......	......	17.80	17.98	18.16
210	......	......	......	......	......	......	......	......	......	......	......	......	18.10	18.30	18.50
215	......	......	......	......	......	......	......	......	......	......	......	......	18.40	18.62	18.83
220	......	......	......	......	......	......	......	......	......	......	......	......	18.70	18.94	19.17

Table V. Dead-Load of Radial-Brick Chimneys in Tons of 2 000 Pounds *

leight n feet	Inside diameter at top, in feet							
	3	4	5	6	7	8	9	10
90	90	98	110	122	......	....	.....	......
100	110	120	131	143	161	180	.....	.....
110	138	143	155	167	188	206	.....	.....
120	160	170	185	198	218	237	.....	.....
130	.....	202	218	231	252	273	295	318
40	.....	237	256	270	290	310	337	357
50	.....	277	296	311	330	353	375	400
60	.....	317	340	357	375	402	423	447
70	.....	362	388	410	425	454	475	500
80	.....	.....	.....	.....	480	510	537	555
90	.....	.....	.....	.....	535	570	592	617
00	.....	.....	.....	.....	600	632	657	685
10	.....	.....	.....	.....	.....	.....	727	760
20	.....	.....	.....	.....	.....	.....	804	843

'hese values are interpolated from curves by the M. W. Kellogg Company, and are und radial-brick chimneys exclusive of the weight of the foundation.

Table VI. Width of Foundations at Base for Radial-Brick Chimneys *

ght eet	Inside diameter at top, in feet							
	3	4	5	6	7	8	9	10
	ft in	ft in	ft in	ft in	ft in	ft in	ft in	ft in
	11 6	12 0	13 0	13 9	.....	.....	.....	.....
	12 6	13 0	14 0	14 8	15 6	16 0	.....	.....
	13 6	14 3	15 0	15 6	16 6	17 0	.....	.....
	14 6	15 3	16 0	16 6	17 6	18 0	.....	.....
	15 6	16 6	17 3	17 8	18 8	19 3	20 0	21 3
	.....	17 9	18 6	18 10	19 9	20 3	21 0	22 3
	.....	19 0	19 9	20 0	21 0	21 6	22 3	23 6
	.....	20 6	21 0	21 6	22 6	23 0	23 6	24 9
	.....	22 0	22 6	23 0	23 9	24 3	25 0	26 3
	.....	.....	.....	.....	25 6	26 0	26 6	27 6
	.....	.....	.....	.....	27 0	27 6	28 3	29 3
	.....	.....	.....	.....	28 6	29 0	29 9	30 9
	.....	.....	.....	.....	.....	.....	31 6	32 6
	.....	.....	.....	.....	.....	.....	33 0	34 3
	.....	.....	.....	.....	.....	.....	.....	37 0

ese values are interpolated from curves by the M. W. Kellogg Company. The n unit soil-pressure at the outer edge of the foundation, due to dead and wind-es not exceed 2 tons per square foot.

forced-Concrete Chimneys. Area of Steel Reinforcement Re- The following formulas are used by one concern in the design of rein-oncrete chimneys:

= wind-moment at section considered, in inch-pounds;

= weight of shell above section considered, in pounds;

D = outside diameter of shell, in feet;
d = inside diameter of shell, in feet;
R = radius of steel circle, in inches;
r = radius of neutral core = $\frac{1}{8}D[1 + (d/D)^2]$ feet;
WR = moment of stability from weight of shell, in inch-pounds;
S = 16 000, the allowable fiber-stress in the steel in pounds per square in
A = total cross-sectional area of steel rods required at section considered, square inches;
$A = 2(M - Wr)/SR$ = number of bars × cross-sectional area of one ba

The bars used are ordinarily ¾-in square, twisted bars, each with a cro sectional area of 0.5625 sq in. The thickness of the shell, if made without taper, is ordinarily 6 in, and if constructed with a taper the shell is made 5 in thickness at the top and increased ¼ in in thickness for each 5 ft in heig The maximum compression due to the wind-moment and dead load in t concrete, at the base, is ordinarily limited to 350 lb per sq in. The same formu apply in this case as in the design of brick chimneys (Fig. 1).

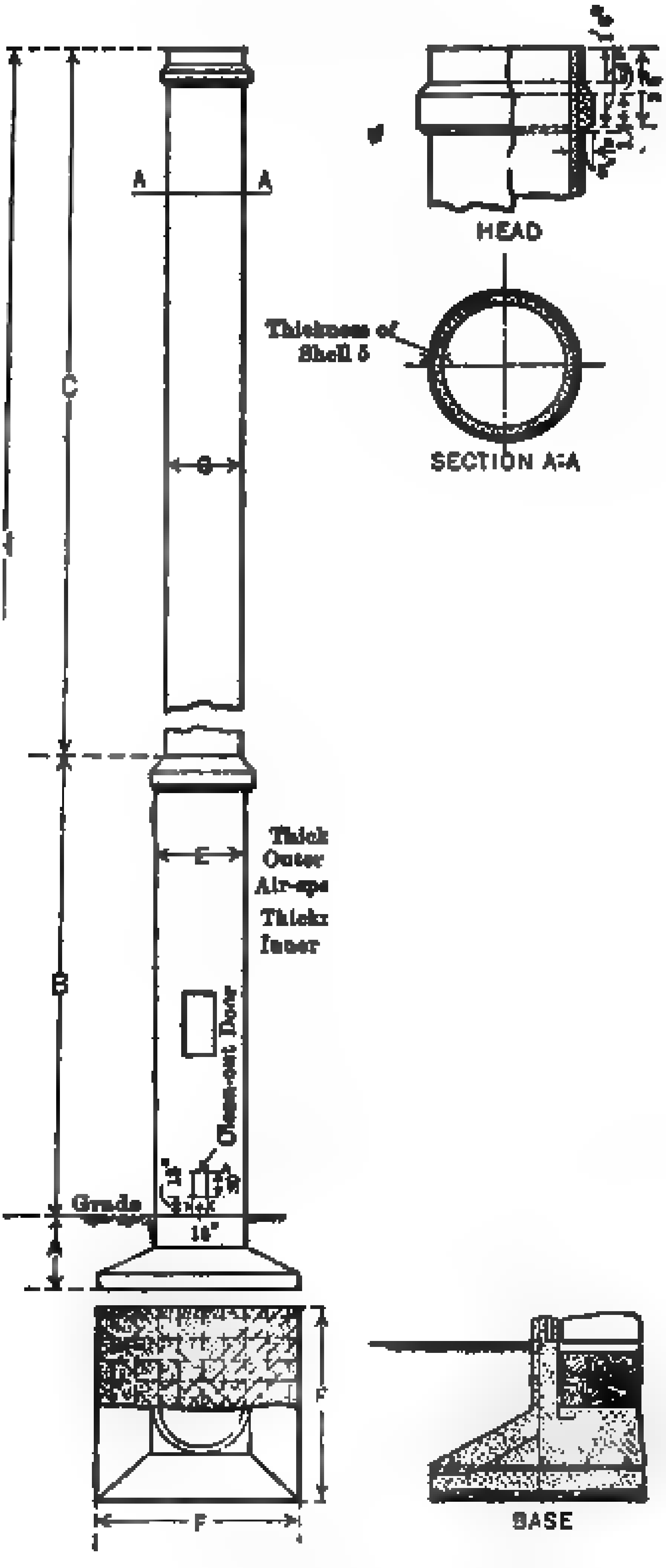
Example. A reinforced-concrete chimney has the following dimensio 150 ft high; no taper; 9 ft inside diameter; thickness of shell 6 in; outsi diameter 10 ft; weight of shell 335 250 lb. It is required to determine the to cross-sectional area of reinforcement.

$M = 25 \times 10 \times 150 \times 150/2 = 2\,812\,500$ ft-lb $= 33\,750\,000$ in-lb;
$r = 10/8 \times [1 + (9/10)^2] = 2.3$ ft $= 27.6$ in;
$R = 58$ in;
$A = 2[33\,750\,000 - (335\,250 \times 27.6)]/16\,000 \times 58) = 53$ sq in;

If ¾-in square bars are used, it requires $53/0.5625 = 94$ bars.

Table VII. Reinforced-Concrete Chimneys. Dimensions. (Fig. 3)

Height above grade ft	Inside diameter ft	Depth below grade ft	Height double shell ft	Height single shell ft	Ttoal height A+B+C ft	Maximum outside diameter ft	in	Width of square foundation ft
H	C	A	B	C	D	E		F
100	4	5	33	67	105	6	4	12
100	5	5	33	67	105	7	4	12
125	5	5	42	83	130	7	4	15
125	6	5	42	83	130	8	4	16
150	6	6	48	102	156	8	4	18
150	7	6	48	102	156	9	4	18
150	8	6	48	102	156	10	4	19
175	8	7	57	118	182	10	6	22
175	9	7	57	118	182	11	6	22
175	10	7	57	118	182	12	6	23
200	10	7	66	134	207	12	6	25
200	11	7	66	134	207	13	6	25
200	12	7	66	134	207	14	6	26
225	12	8	69	156	233	14	8	29
225	13	8	69	156	233	15	8	29
225	14	8	69	156	233	16	8	30
250	14	8	81	169	258	16	8	32
250	15	8	81	169	258	17	8	33
250	16	8	81	169	258	18	8	34



ig. 3. Details of Tall Reinforced-concrete Chimney

himney Company, Chicago, Ill., designed and constructed, chimneys, the great reinforced-concrete chimney at Saganoseki, riental Compressol Company, for the copper smelter. It was uary, 1917, and ranks with the highest in the world, being foundations and 26¼ ft in internal diameter at the top.

Self-Sustaining Steel Chimneys* are largely used, especially for tall chimneys of iron-works and power-houses from 150 to 300 ft in height.

"The advantages claimed are: Greater strength and safety; smaller space required; smaller cost by 30 to 50% as compared with brick chimneys; avoidance of infiltration of air and consequent checking of the draught, common in brick chimneys. They are usually made cylindrical in shape, with a wide curved flare for 10 to 25 ft at the bottom. A heavy cast-iron base-plate is provided, to which the chimney is riveted, and the plate is secured to a massive foundation by holding-down bolts. No guys are used."†

The largest SELF-SUSTAINING STEEL CHIMNEY in the world (1919) is that built by the Chicago Bridge and Iron Works at the plant of the United Verde Copper Company, Clarkdale, Arizona. It is 30 ft 9½ in in diameter and 400 ft in height. The thickness of plates varies from ¼ in at the top to 11/16 in for the bell-shaped portion at the bottom. The weight of steel is 800 000 lb. The stack is anchored to the foundation by thirty-six bolts, each 4 in in diameter, upset, and spaced equidistant in a bolt-circle of 25 ft 4½ in radius.

Table VIII. Sizes of Foundations for Self-Sustaining Steel Chimneys, Half-Lined‡

Diameter, clear, in feet...	3	4	5	6	7	9	11
	ft in	ft in	ft in	ft in	ft in	ft in	ft in
Height................	100	100	150	150	150	175	225
Least diam. of foundation	15 9	15 3	20 4	21 10	22 7	25 9	29 11
Least depth of foundation	6 6	7	9	8	9	10	13
Height................	....	125	200	200	250	275	300
Least diam. of foundation	....	17 6	23 8	25 0	29 8	33 6	36 0
Least depth of foundation	....	7 6	10	10	12	12	14

The governing feature in the design of a SELF-SUSTAINING STEEL CHIMNEY OR STACK is the force of the wind. The cylinder above any horizontal plane section may be assumed to act as a cantilever beam in which the bending moment, in foot-pounds, is

$$M = HD \times P \times \tfrac{1}{2}H = \tfrac{1}{2}H^2DP$$

in which H is the height in feet above the section considered, D the diameter in feet and P the assumed pressure of the wind in pounds per square foot on a vertical cross-section. The fiber-stress S, in pounds per square inch, according to the formula for flexure, is $S = Mc/I$. For hollow cylinders of large diameter and small thickness, the moment of inertia $I = \pi R^3 t$, in which R = mean radius in feet (equivalent to c in the flexure formula) and t = thickness of shell in inches. Hence

$$S = MR/12\pi R^3 t = 0.106M/D^2 t, \quad \text{and} \quad t = 0.106M/SD^2.$$

The stress S is tensile on the windward side and compressive on the leeward side.

* Compiled from data furnished by Robins Fleming.

† Mechanical Engineers' Pocket Book. Kent.

‡ These dimensions were taken from a pamphlet published by the Philadelphia Engineering Works.

small compressive stress due to the weight of the stack. The
be taken at 25 lb per sq ft and of S at 16 000 lb per sq in, as given
tions for the Structural Steel Work for Buildings, Chapter XXX.
ouilt for durability as well as strength it is often advisable to
oretical thickness of the shell. No plate should be used with a
han ¼ in. It is important that the stack be securely anchored
n. Many methods have been proposed for determining stresses
As the problem depends for its solution on the physical con-
base and bolts, no exact analysis is possible. (See editorial dis-
icle, Anchor-Bolt Tension, in Engineering News, April 30, 1914.)
assumption is that the bolts are screwed up with a high initial
nchor-bolt ring can then be considered in the same way as a ring
The maximum stress at any point of the bolt-circle is devel-
wind is blowing parallel to the radius through that point. The
ircumferential inch is $0.106M/(2R_1)^2$, $2R_1$ being the diameter
le. Let b be the circumferential distance in inches between
-bolts, N the number of bolts equidistant on the bolt-circle and
the stack. For the anchor-bolt on the windward side there is a
, due to the wind,

$$S_w = 0.106bM/(2R_1)^2$$

$$b = (2R_1 \times 12 \times \pi)/N$$

$$S_w = 2M/R_1N$$

eight of the portion of the stack between adjacent bolts, the
stress in any anchor-bolt may be expressed by the equation

$$S_w = (2M/R_1N) - W/N$$

Chimneys. These chimneys are built with special blocks
ne circular and radial lines of each section of the chimney so
brickwork has joints of an even thickness throughout and a
surface. The blocks being much larger than common bricks,
om one third to one half as many joints. Radial-brick chim-
ircular in plan above the base. The best form of base is octag-
ion so as to permit the breeching to enter the chimney at a
at the same time comply best with the rules of stability.
cal-works, refineries, furnaces, etc., radial-brick chimneys are
e shell, a lining only being provided in the immediate vicinity
ce. All radial bricks are perforated vertically and this insures
and allows the mortar to enter the perforations, thus forming
ige.
for chimney-construction have been used extensively in
y, France and Russia since 1870. They were not introduced
however, until 1898. About forty-five years ago (1869 or
stodis, of Düsseldorf, Germany, originated a method of building
erforated radial blocks, made from selected clays and burned
mperature, and in 1898 an American company * was formed
f erecting chimneys by this method of construction. Since

stodis Chimney Construction Company, New York City.

that time the company through various agencies has built more than six thousand chimneys in all parts of the world. The tallest chimney in the world (1919), 585 feet high and 60 ft in internal diameter at the top, was built by this company in 1918 for the Anaconda Copper Company, at Anaconda, Mont.

Mr. H. R. Heinicke,* of Chemnitz, Germany, builder of the 460-ft stack at Halsbrücke, Germany, has employed radial bricks made especially for each chimney. This firm through long and costly research has done much to make chimney-building a science. The chimney at Halsbrücke is a very remarkable one on account of its proportions. In a height of 460 ft, the diameter at the top is only 8 ft, whereas the 585-ft stack at Anaconda, Mont., has a diameter of 60 ft at the top.

The Heine Chimney Company † has erected many important high chimneys. The essential difference in the methods of construction used by this company from those of the other chimney-constructors is that the Heine Chimney Company uses perforated, INTERLOCKING, radial bricks. It is claimed that this interlocking-feature has an advantage over the straight-sided bricks in acting as a preventive of deep weathering of the joints and of air-leaks. In addition to this it is claimed that the circumferential strength of the walls when built of this type of brick is considerably greater than when built with plain-sided or corrugated bricks. The perforations in these bricks are fewer but larger than those of some of the other constructors. The brickwork is laid on full-mortar beds with SHOVED joints. These large perforations allow the mortar to rise in them, thus forming PINS which give the walls great strength and enable them to withstand the stresses due to expansion caused by the high temperature of the flue-gases. In walls more than one brick thick, the bricks are laid up in English bond, that is, with alternate header and stretcher-courses. This company advocates this method of construction even in chimneys built with the ordinary straight-sided common building-bricks. Among the many important chimneys constructed by the Heine Chimney Company is the one erected at the St. Joseph Lead Company's plant, at Herculaneum, Mo. The height of this chimney is 350 ft and the inside diameter at the top 20 ft. (See page 1706.)

The M. W. Kellogg Company ‡ has designed and built many radial-brick chimneys for power-plants, chemical-works and other purposes. Several of the important chimneys put up by them are mentioned in the list of tall chimneys (page 1379). Some of the details of construction differ from those of the other companies mentioned. One of the points of difference is the detail relating to the corrugations on their bricks. These corrugations are 3/8 in wide and 1/4 in deep and are placed along the vertical sides of the bricks as they lie in the wall. The adhesion between the bricks and mortar is increased by this increased area. It is claimed that tests made show that this is the case. On account of these corrugations it is not considered necessary to embed any ironwork in these chimneys to prevent the development of cracks due to heat-expansion. Ironwork has sometimes been inserted when plain-sided bricks have been used. It is claimed that this design is somewhat heavier than that employed by some other constructors, this company holding that it is not safe to figure on wind pressure of less than 25 lb per sq ft of projected area. Among the many tall chimneys erected by this company may be mentioned especially the

* H. R. Heinicke, Incorporated, New York City.
† The Heine Chimney Company, Chicago, Ill.
‡ The M. W. Kellogg Company, New York City.

glas, Ariz., erected for the Copper Queen Consolidated Mining

er reliable companies which design and construct tall chimneys.
d here were the pioneers in this work.

List of Tall Chimneys Over 300 Feet in Height

)ted that this list is constantly added to from year to year.'

	Height, ft	Diam. inside at top, ft
ıt., Anaconda Copper Co. (1918).............	585	60
., American Smelting & Refining Co. (1917)...	573	25
pan, Oriental Compressol Co. (1917)..........	570	$26\frac{1}{8}$
[ont., Boston & Montana Consolidated Copper ining Co. (1907)...........................	506	50
ıy, Germany, Halsbrücke Foundry............	460	8
)undas, Scotland, F. Townsend...............	454	..
)llox, Scotland, Tenant & Co................	$436\frac{1}{2}$	..
Jnited Verde Extension Mining Co. (1918).....	425	30
:, Messrs. Musprath Chemical Works..........	406	..
, United Verde Copper Co...................	400	$30\frac{5}{8}$
Consolidated Kansas City Smelting & Refining	400	30
American Smelting & Refining Co. (1911)......	400	25
ont., American Smelting & Refining Co. (1917).	400	16
Clough Mill, Scotland, Messrs. Crossley's.......	381	..
K. Williams & Co. (1911)..................	375	7
ton, England, Dobson & Barlow..............	367	..
., Eastman Kodak Co. (two) (1906, 1911).....	366,	9 and 13
, N. J., Orford Copper Co. (two) (1900, 1910)..	365	10
Garfield Smelting Co. (1913)................	350	22
Mo., St. Joseph, Lead Co....................	350	20
Fall River Iron Co.........................	350	11
Heller Merz Co (1904)......................	350	8
[. J., Clark Thread Co......................	335	..
ı, Germany, Wessenfield & Co..............	331	..
land, Gas-Works..........................	329	..
nn., Tennessee Copper Co..................	325	20
d., Indianapolis Traction Co................	320	13
ıgland, Brook & Son, Fire-clay Works........	315	..
land, Adams Soap-Works...................	312	..
., Rhode Island Suburban Railway Co........	308	16
N. Y., New York Steam Co. (1904)..........	308	15
d, P. Dickdn & Son.......................	300	..
nd, Mitchell Brothers.....................	300	..

' the Alphons Custodis Chimney Construction Company, New York

:rete, The Weber Chimney Company, Chicago, Ill.
H. R. Heinicke, Incorporated, New York City.
steel chimney, the largest (of this type) in the world (1919).
The Heine Chimney Company, Chicago, Ill.

Partial List of Tall Chimneys over 300 Feet in Height (Continued)

	Height ft	Diam. inside at top ft
*Garfield, Utah, American Smelting and Refining Co. (1905)....	300	30
*Hayden, Ariz., American Smelting and Refining Co...........	300	25
† Douglas, Ariz. Copper Queen Consolidated Mining Co........	300	22
‡Tacoma, Wash., Tacoma Smelting Co........................	300	18
§McGill, Nev., Steptoe Valley Traction Co....................	300	15
*Brooklyn, N. Y., Nichols Chemical Co. (1905)...............	300	12
*Claymont, Del., General Chemical Co. (1912)................	300	8

* Constructed by the Alphons Custodis Chimney Construction Company, New York City.
† Constructed by The M. W. Kellogg, Company, New York City.
‡ Reinforced concrete, The Weber Chimney Company, Chicago, Ill.
§ Constructed by H. R. Heinicke, Incorporated, New York City.

LICS, PLUMBING AND DRAINAGE, ILLUMINATING-GAS AND GAS-PIPING

BY
J. J. COSGROVE
CONSULTING SANITARY ENGINEER

(1) HYDRAULICS

ctically an incompressible liquid, weighing, at the average temper-
62.355 lb to the cu ft and 8.335 lb to the gallon. These figures
with changes in temperature and atmospheric pressure, and a
for the same temperature will be found in different works.

Water. The pressure of still water in pounds per square inch
s of any pipe or vessel of any shape whatever is due alone to the
of the surface of the water above the point considered pressed
qual to 0.433 lb per sq in for every foot of head at 62° F. The
er square inch is equal in all directions. To find the total pres-
ater against and perpendicular to any surface, whether vertical,
inclined at any angle, whether it be flat or curved, multiply
ea in square feet of the surface pressed, the vertical depth of its
ty below the surface of the water, and the constant 62.4. The
the required pressure in pounds. This may be expressed by
ws:

$$P = 62.4\, AD$$

essure in pounds of quiescent water on the surface considered;
ea pressed upon in square feet; and
rtical depth in feet of center of gravity of surface considered.

Pressure in Pounds per Square Inch for Different Heads of Water in Feet

1	2	3	4	5	6	7	8	9
0.433	0.866	1.299	1.732	2.165	2.598	3.031	3.464	3.897
4.763	5.196	5.629	6.062	6.495	6.928	7.361	7.794	8.227
9.093	9.526	9.959	10.392	10.825	11.258	11.691	12.124	12.557
13.423	13.856	14.289	14.722	15.155	15.588	16.021	16.454	16.887
17.753	18.186	18.619	19.052	19.485	19.918	20.351	20.784	21.217
22.083	22.516	22.949	23.382	23.815	24.248	24.681	25.114	25.547
26.413	26.846	27.279	27.712	28.145	28.578	29.011	29.444	29.877
30.743	31.176	31.609	32.042	32.475	32.908	33.341	33.774	34.207
35.073	35.506	35.939	36.372	36.805	37.238	37.671	38.104	38.537
39.403	39.836	40.269	40.702	41.135	41.568	42.001	42.436	42.867

for greater heads can be readily found by multiplication or addi-
pressure for a head of 110 ft is ten times that for 11 ft. The pres-
s equal to the pressure for 110 ft plus that for 8 ft.

Flow of Water in Pipes. Owing to the many practical and variable conditions which affect the flow of water in pipes, such as the smoothness of the pipe, number and character of the joints, bends and valves in the pipe, to say nothing of the size and length of the pipe, all formulas for the velocity and discharge of water in and through pipes can only be considered as approximate. The following formulas and data are taken largely from the National Tube Company's Book of Standards, 1902 edition. They agree fairly well with similar tables by Kent and Trautwine, both of whom devote much space to this subject. The QUANTITY OF WATER passing through a given pipe is governed by the sectional area of the pipe or outlet and the mean VELOCITY. The velocity depends primarily upon the PRESSURE or HEAD, and is greatly affected by FRICTION, which again varies with the smoothness of the bore, the diameter and length of the pipe, and whatever obstructions there may be in the pipe. The HEAD is the vertical distance from the surface of the water in the reservoir to the center of gravity of the lower end of the pipe when the discharge is into the air, or to the level surface of the lower reservoir when the discharge is under water. When the pressure is produced by mechanical means, the head of water in feet may be readily determined by the following table:

Table B.* For Converting Pressure in Pounds per Square Inch into Head of Water in Feet

Pressure	0	1	2	3	4	5	6	7	8	9
0	......	2.309	4.619	6.928	9.238	11.547	13.857	16.166	18.476	20.785
10	23.0947	25.404	27.714	30.023	32.333	34.642	36 952	39.261	41.570	43.880
20	46.1894	48.499	50.808	53.118	55.427	57.737	60.046	62.356	64.665	66.975
30	69.2841	71.594	73.903	76.213	78.522	80.831	83.141	85.450	87.760	90.069
40	92.3788	94.688	96.998	99.307	101.62	103.93	106.24	108.55	110.85	113.16
50	115.4735	117.78	120.09	122.40	124.71	126.02	129.33	131.64	133.95	136.26
60	138.5682	140.88	143.19	145.50	147.81	150.12	152.42	154.73	157.04	159.35
70	161.6629	163.97	166.28	168.59	170.90	173.21	175.52	177.83	180.14	182 45
80	184.7576	187.07	189.38	191.69	194.00	196.31	198.61	200.92	203.23	205.54
90	207.8523	210.16	212.47	214.78	217.09	219.40	221.71	224.02	226.33	228.64

* Tables A and B are exact for water at 62° F. and for atmospheric pressure at 14.7 lb per sq in.

To find the velocity of water discharged from a pipe-line longer than four times its diameter, knowing the head, length and inside diameter, use the following formula:

$$v = m\sqrt{\frac{hd}{L + 54d}}$$

in which

- v = approximate mean velocity in feet per second;
- m = coefficient from the table below;
- d = diameter of pipe in feet;
- h = total head in feet;
- L = total length of line in feet.

The following coefficients are averages deduced from a large number of experiments. In most cases of pipes carefully laid and in fair condition, they should give results varying not more than from 5 to 10%.

Values of Coefficient m

Diameter of pipe in feet							
0.05	0.10	0.50	1	1.5	2	3	4
m	m	m	m	m	m	m	m
29	31	33	35	37	40	44	47
34	35	37	39	42	45	49	53
39	40	42	45	49	52	56	59
41	43	47	50	54	57	60	63
44	47	52	54	56	60	64	67
47	50	54	56	58	62	66	70
48	51	55	58	60	64	67	70

Given the head, $h = 50$ ft; the length, $L = 5\,280$ ft and the diam-
t; to find the velocity and quantity of discharge.
ing these values in the foregoing formula, we get

$$\sqrt{\frac{d \times h}{L + 54\,d}} = \sqrt{\frac{2 \times 50}{5\,280 + 108}} = \sqrt{\frac{100}{5\,388}} = 0.136$$

ın headed $\sqrt{\dfrac{hd}{L + 54\,d}}$ find 0.10, which is the value nearest to 0.136, along this line until column headed 2 is reached; then read 62 as of coefficient m.

$= 62 \times 0.136 = 8.432$ ft per sec, the velocity required.

d the discharge in cubic feet per second, multiply this velocity by oss-section of pipe in square feet.

ʒ.1416 $\times (1)^2 \times 8.432 = 26.49$ cu ft per sec.

here are 7.48 gal in a cubic foot, the discharge in gallons per second = '.48 = 198.2.

bove formula is only an approximation, since the flow is modified by ›ints, incrustations, etc.

ıd the head in feet necessary to give a stated discharge in cubic feet, formula

$$h = \frac{0.000704\,Q^2\,(L + 54\,d)}{d^5}$$

h

= total head in feet;
= total length of line in feet;
= diameter of pipe in feet;
= quantity of water in cu ft per second.

mple. Given the diameter of pipe, $d = 0.5$ ft; the length of pipe, $L =$ and the quantity of water to be discharged, $q = 3.07$ cu ft per sec; to he necessary head.

›stituting these values in the above formula, we get

$$h = \frac{0.000704 \times 9.4 \times (20 + 27)}{(0.5)^5}$$

$$= \frac{0.000704 \times 9.4 \times 47}{.0.03125} = 9.95 \text{ ft, the required head.}$$

The following formula is simpler and can be used when 54 d in relation to L is so small as to be negligible:

$$h = \frac{0.000704\, Q^2 \times L}{d^5}$$

If the pipe instead of being straight has easy curves (say with radius not less than five diameters of the pipe) either horizontal or vertical, the discharge will not be materially diminished so long as the total heads and total actual lengths of pipe remain the same, but it is advisable to make the radius as much more than five diameters as can conveniently be done.

To find the diameter of a pipe of given length to deliver a given quantity of water under a given head use the following,

$$d = 0.234 \sqrt[5]{\frac{Q^2 L}{h}}$$

in which

d = diameter of pipe in feet;
Q = cubic feet per second delivered;
L = length of line in feet;
h = head in feet.

Example. Given the head, $h = 700$ ft; the length of pipe, $L = 3\,000$ ft; the quantity to be delivered, $Q = 4$ cu ft per sec; required the diameter of pipe necessary.

Substituting these values in the foregoing formula, we get:

$$d = 0.234 \sqrt[5]{\frac{16 \times 3\,000}{700}} = 0.234 \sqrt[5]{68.57} = 0.545 \text{ ft} = 6.54 \text{ in}$$

To find the diameter of pipe required to deliver a given quantity of water with a given head.

Rule. (1) Reduce the head to feet per 100 ft; (2) from Table C, page 1385, find the discharge for the head thus obtained through a pipe 1 ft in diameter; (3) divide the required discharge by that obtained from Table C; look for the quotient in the column of Table D, page 1386, headed Ratio of Discharge, etc., and opposite it, in the adjoining columns of the table, will be found the required diameter.

Note. The use of Tables C and D gives results sufficiently correct for pipes less than 700 diameters in length.

Example. If the head of water from a reservoir to the point of delivery is 20 ft in a distance of 1 860 ft, what is the diameter of a pipe required to deliver 6 cu ft of water per second?

20 ft head in 1 860 ft = 20/18.60 ft in 100 ft, or 1.075 ft in 100

From Table C we find that the discharge per second with a head of 1.136 is 3.989 cu ft; for a head of 1.075 it would be about 3.8 cu ft. Dividing the required discharge 6, by 3.8 cu ft per sec, we have 1.58. From Table D the diameter of pipe having a ratio of discharge equal to 1.58 is found to be about 14½ in; therefore we must use a 15-in pipe to obtain the required discharge. If the required discharge is in gallons, divide by 7.5 to reduce to cubic feet. If in cubic feet per minute, divide by 60 to reduce to feet per second.

ocities and Discharges Through a Straight, Smooth Pipe One Diameter and One Mile, or 5 280 Diameters, in Length

	Head in feet per mile	Velocity in feet per sec	Discharge in cubic feet per sec	Discharge in cubic feet per 24 hours
	3	1.13	0.8914	76 982
	4	1.31	1.028	88 862
	5	1.47	1.150	99 403
	6	1.61	1.264	109 209
	7	1.74	1.366	118 022
	8	1.86	1.455	125 740
	9	1.96	1.539	132 969
	10	2.08	1.633	141 145
	12	2.27	1.782	153 964
	14	2.45	1.924	166 233
	16	2.62	2.057	177 724
	18	2.78	2.183	188 611
	20	2.93	2.301	198 806
	25	3.28	2.572	222 156
	30	3.59	2.819	243 604
	35	3.88	3.047	263 260
	40	4.15	3.267	282 288
	45	4.40	3.451	298 209
	50	4.64	3.638	314 352
	60	5.08	3.989	344 649
	70	5.49	4.311	372 470
	80	5.85	4.602	397 613
	90	6.23	4.900	423 435
	100	6.56	5.144	444 312
	110	6.87	5.395	466 128
	120	7.18	5.639	487 209
	130	7.47	5.866	506 822
	140	7.76	6.094	526 521
	150	8.05	6.322	546 048
	160	8.30	6.534	564 576
	170	8.55	6.715	580 176
3	180	8.80	6.903	596 418
5	190	9.04	7.100	613 440
8	200	9.28	7.276	628 704
1	225	9.84	7.696	664 848
5	250	10.4	8.168	705 728
8	275	10.8	8.482	732 844
2	300	11.3	8.914	769 824
9	350	12.3	9.621	831 168
76	400	13.1	10.28	888 624
32	450	13.9	10.91	943 056
7	500	14.7	11.50	994 032
1	550	15.4	12.09	1 044 576
6	600	16.1	12.64	1 092 096
0	650	16.7	13.11	1 132 704
25	700	17.4	13.66	1 180 224
20	750	18.0	14.13	1 220 832
.15	800	18.6	14.55	1 257 408
.09	850	19.1	15.00	1 296 000
.04	900	19.6	15.39	1 329 696
.99	950	20.3	15.94	1 377 216
.94	1 000	20.8	16.33	1 411 456
.73	1 200	22.7	17.82	1 539 648
.52	1 400	24.5	19.24	1 662 336
.30	1 600	26.2	20.57	1 777 248
.08	1 800	27.8	21.83	1 886 112
7.87	2 000	29.3	23.01	1 988 064
7.35	2 500	32.8	25.72	2 221 560
6.81	3 000	35.9	28.19	2 436 040

Table D. Diameters of Pipes and Ratio of Discharge

Diameter of pipe, in	Diameter of pipe, ft	Ratio of discharge to that through a 1-ft pipe with the same head per mile	Diameter of pipe, in	Diameter of pipe, ft	Ratio of discharge to that through a 1-ft pipe with the same head per mile
1	0.0833	0.0020	12½	1.042	1.106
1½	0.1250	0.0055	13	1.083	1.221
2	0.1667	0.0113	14	1.167	1.470
2½	0.2083	0.0198	15	1.250	1.746
3	0.2500	0.0310	16	1.333	2.053
3½	0.2917	0.0458	17	1.417	2.388
4	0.3333	0.0643	18	1.5	2.754
4½	0.3750	0.0857	19	1.583	3.153
5	0.4167	0.1119	20	1.667	3.585
5½	0.4583	0.1422	21	1.75	4.051
6	0.5	0.1767	22	1.833	4.551
6½	0.5417	0.2159	23	1.917	5.084
7	0.5833	0.2600	24	2	5.649
7½	0.6250	0.3090	24⅝	2.052	6.000
8	0.6667	0.3631	26	2.167	6.912
8½	0.7083	0.4220	28	2.333	8.319
9	0.75	0.4871	30	2.5	9.822
9½	0.7917	0.5575	30¼	2.521	10.0
10	0.8333	0.6337	32	2.667	11.6
10½	0.8750	0.7157	34	2.833	13.5
11	0.9167	0.8044	36	3	15.5
11½	0.9583	0.8987	38	3.167	17.8
12	1	1	40	3.333	20.2

This table shows, also, the relative discharging capacities of long pipes. Thus, one 12-in pipe is equal to two 9-in pipes, to nearly six 6-in pipes, or to thirty-three 3-in pipes.

Table E. Flow of Water in House Service-Pipes

Thomson Meter Company

To find the discharge in gallons, multiply by 7.47

Condition of discharge	Pressure in main, lb per sq in	Discharge in cubic feet per minute from the pipe								
		Nominal diameters of iron or lead service-pipe in inches								
		½	⅝	¾	1	1½	2	3	4	6
Through 35 ft of service-pipe; no back-pressure	30	1.10	1.92	3.01	6.13	16.58	33.34	88.16	173.85	444.63
	40	1.27	2.22	3.48	7.08	19.14	38.50	101.80	200.75	513.42
	50	1.42	2.48	3.89	7.92	21.40	43.04	113.82	224.44	574.02
	60	1.56	2.71	4.26	8.67	23.44	47.15	124.68	245.87	628.81
	75	1.74	3.03	4.77	9.70	26.21	52.71	139.39	274.89	703.03
	100	2.01	3.50	5.50	11.20	30.27	60.87	160.96	317.41	811.79
	130	2.29	3.99	6.28	12.77	34.51	69.40	183.52	361.91	925.58
'hrough 100 ft of service-pipe; no back-pressure	30	0.66	1.16	1.84	3.78	10.40	21.30	58.19	118.13	317.23
	40	0.77	1.34	2.12	4.36	12.01	24.59	67.19	136.41	366.30
	50	0.86	1.50	2.37	4.88	13.43	27.50	75.13	152.51	409.54
	60	0.94	1.65	2.60	5.34	14.71	30.12	82.30	167.06	448.63
	75	1.05	1.84	2.91	5.97	16.45	33.68	92.01	186.78	501.58
	100	1.22	2.13	3.36	6.90	18.99	38.89	106.24	215.68	579.18
	130	1.39	2.42	3.83	7.86	21.66	44.34	121.14	245.91	660.36
hrough 100 ft of service-pipe and 15-ft ver-:ical rise	30	0.55	0.96	1.52	3.11	8.57	17.55	47.90	97.17	260.56
	40	0.66	1.15	1.81	3.72	10.24	20.95	57.20	116.01	311.09
	50	0.75	1.31	2.06	4.24	11.67	23.87	65.18	132.20	354.49
	60	0.83	1.45	2.29	4.70	12.94	26.48	72.28	146.61	393.13
	75	0.94	1.64	2.59	5.32	14.64	29.96	81.79	165.90	444.85
	100	1.10	1.92	3.02	6.21	17.10	35.00	95.55	193.82	519.72
	130	1.26	2.20	3.48	7.14	19.66	40.23	109.82	222.75	597.31
rough oo ft of ervice-ipe and o-ft ver-ical rise	30	0.44	0.77	1.22	2.50	6.80	14.11	38.63	78.54	211.54
	40	0.55	0.97	1.53	3.15	8.68	17.79	48.68	98.98	266.59
	50	0.65	1.14	1.79	3.69	10.16	20.82	56.98	115.87	312.08
	60	0.73	1.28	2.02	4.15	11.45	23.47	64.22	130.59	351.73
	75	0.84	1.47	2.32	4.77	13.15	26.95	73.76	149.99	403.98
	100	1.00	1.74	2.75	5.65	15.58	31.93	87.38	177.67	478.55
	130	1.15	2.02	3.19	6.55	18.07	37.02	101.33	206.04	554.96

ble E may also be used when the pressure is in feet-head of water by reducing ead in feet to pounds per square inch by Table A. Thus, if we wish the arge per minute through a ¾-in pipe 100 ft long with a head of 70 ft, we rom Table A that a head of 70 ft corresponds to a pressure of 30 lb per sq d from Table E we find the discharge through a ¾-in pipe 100 ft long with ssure of 30 lb to be 1.84 cu ft per minute.

Table F. Friction of Water in Pipes Based on Ellis and Howland's Experiments

The following table gives the friction-loss in pounds-pressure per square inch for EACH 100 ft of length in clean iron pipes of different sizes, discharging given quantities of water per minute. This friction-loss is greatly increased by bends or irregularities in the pipe.

To find the friction-head in feet, multiply by 2.3

Gallons per minute	Sizes of pipes, inside diameter							
	¾ in	1 in	1¼ in	1½ in	2 in	2½ in	3 in	4 in
5	3.3	0.84	0.31	0.12	......	......	......	......
10	13.0	3.16	1.05	0.47	0.12	......	......	......
15	28.7	6.98	2.38	0.97	0.26	......	......	......
20	50.4	12.3	4.07	1.66	0.42	......	......	......
25	78.8	19.0	6.40	2.62	0.64	0.21	0.10	0.27
30	......	27.5	9.15	3.75	0.91	......	......	......
35	......	37.0	12.4	5.05	1.22	......	......	......
40	......	48.0	16.1	6.52	1.60	......	0.20	......
45	......	......	20.2	8.15	2.02	......	......	......
50	......	......	24.9	10.0	2.44	0.81	0.35	0.09
75	......	......	56.1	22.4	5.32	1.80	0.74	0.23
100	......	......	......	39.0	9.46	3.20	1.31	0.33
125	......	......	......	......	14.9	4.89	1.99	0.49
150	......	......	......	......	21.2	7.00	2.85	0.69
175	......	......	......	......	28.1	9.46	3.85	0.94
200	......	......	......	......	37.5	12.47	5.02	1.22
250	......	......	......	......	......	19.66	7.76	1.89
300	......	......	......	......	......	28.06	11.2	2.66
350	......	......	......	......	......	......	15.2	3.65
400	......	......	......	......	......	......	19.5	4.73
450	......	......	......	......	......	......	25.0	6.01
500	......	......	......	......	......	......	30.8	7.43
600	......	......	......	......	......	......	......	9.54
700	......	......	......	......	......	......	......	14.32

Water-Pipe is usually tested to 300 lb pressure per square inch before delivery, and a hammer-test should be made while the pipe is under pressure. The usual length for each section of cast-iron water-pipe is from 12 ft 4 in to 12 ft 6 in, depending upon the depth of the socket, each length making approximately 12 ft of pipe when laid. Pipes from 2 to 4 in diameter are sometimes made in 8 or 9-ft lengths.

Safe Pressures and Equivalent Heads of Water for Cast-Iron Pipes of Different Sizes and Thicknesses

Calculated by F. H. Lewis from Fanning's Formula

Thickness, in	Size of pipe, in: 4		6		8		10		12		14	
	Pressure, lb	Head, ft	Pressure, lb	Head, ft	Pressure, lb	Head, ft	Pressure, lb	Head, ft	Pressure, lb	Head, ft	Pressure, lb	Head, ft
7/16	112	258	49	112	18	42	....	....	....	....	....	....
1/2	224	516	124	280	74	171	44	101	24	55	....	....
9/16	336	774	199	458	130	300	89	205	62	143	42	97
5/8	...	...	274	631	186	429	132	304	99	228	74	170
11/16	...	...	...	...	...	...	177	408	137	316	106	244
3/4	...	...	...	...	...	...	224	516	174	401	138	316
13/16	...	...	...	...	...	...	....	....	212	488	170	392
7/8	...	...	...	...	...	...	....	....	249	574	202	465
15/16	...	...	...	...	...	...	....	....	....	....	234	538
1	...	...	...	...	....	...	....	....	....	....	266	612
	16		18		20		24		30		36	
5/8	56	129	41	95	...	...	....	....	....	....	....	....
11/16	84	194	66	152	51	118	30	69	....	....	....	....
3/4	112	258	91	210	74	170	49	113	24	55	....	....
13/16	140	323	116	267	96	221	68	157	39	90	....	....
7/8	168	387	141	325	119	274	86	198	54	124	32	74
15/16	196	452	166	382	141	325	105	242	69	159	44	101
1	224	516	191	440	164	378	124	286	84	194	57	131
1 1/8	...	...	216	497	209	481	161	371	114	263	82	189
1 1/4	...	...	...	...	256	589	199	458	144	332	107	247
1 3/8	...	...	...	...	...	...	237	546	174	401	132	304
1 1/2	...	...	...	...	...	...	....	....	204	470	157	362
1 5/8	...	...	...	...	...	...	....	....	234	538	182	419
1 3/4	...	...	...	...	...	...	....	....	....	....	207	477

Weights of Lead and Gaskets for Pipe-Joints

Dennis Long & Company

Diameter of pipe, in	Lead, lb	Gasket, lb	Diameter of pipe, in	Lead, lb	Gasket, lb
2	2.5	0.125	12	15	0.250
3	3.5	0.170	14	18	0.375
4	4.5	0.170	16	22	0.500
6	6.5	0.200	18	26	0.500
8	9.0	0.200	20	33	0.625
10	13.0	0.250	....	....	....

Weights, per Foot, of Cast-Iron Pipes in General Use, Including Socket-Ends and Spigot-ends

Dennis Long & Company, Inc., Louisville, Ky.

Diameter, in	Thickness, in	Weight per ft, lb	Diameter, in	Thickness, in	Weight per ft, lb	Diameter, in	Thickness, in	Weight per ft, lb
3	3/8	12½	16	¾	129	30	2	662
	7/16	15		⅞	152	36	⅞	334
	½	18		1	175		1	382
	9/16	20½	18	⅝	120		1⅛	432
	⅝	23		¾	146		1¼	482
4	⅜	17		⅞	171		1⅜	532
	7/16	20		1	197		1½	587
	½	23½		1⅛	223		1⅝	632
	9/16	26¾		1¼	249		1¾	683
	⅝	30	20	11/16	148		1⅞	734
6	7/16+	30		¾	161		2	786
	½	34		⅞	190	42	1	445
	9/16	38¼		1	216		1⅛	471
	⅝	42½		1⅛	247		1¼	560
	¾	52		1¼	276		1⅜	629
8	7/16	40		1⅜	305		1½	675
	½	43½		1½	334		1⅝	734
	9/16	49¾	24	¾	191		1¾	794
	⅝	56		⅞	225		1⅞	853
	¾	68		1	258		2	912
10	7/16	50		1⅛	293	48	1⅛	572
	½	54		1¼	327		1¼	637
	9/16	60		1⅜	361		1⅜	701
	⅝	68		1½	395		1½	768
	¾	82		1⅝	430		1⅝	835
12	½	70		1¾	465		1¾	901
	9/16	76	30	13/16	258		1⅞	967
	⅝	82		⅞	278		2	1 034
	¾	99		1	319	60	1¼	797
	⅞	117		1⅛	360		1⅜	880
14	9/16	85		1¼	405		1½	964
	⅝	94		1⅜	448		1⅝	1 049
	¾	113		1½	489		1¾	1 133
	⅞	137		1⅝	532		1⅞	1 216
16	9/16	100		1¾	575		2	1 300
	⅝	108		1⅞	619		2¼	1 470

There is no standard weight of pipe for any given pressure.

Private Water-Supply. Pumps

Private Water-Supplies. The architect is frequently required to furni[sh] a water-supply for isolated buildings, and even in cities it is becoming qui[te] common for manufacturing establishments and large buildings to have th[eir] own water-supply; so that some knowledge of the various methods of su[p]plying water is requisite. Power-pumps are of so many kinds and so int[ri]cate in construction that no attempt will be made to describe them.

The Hydraulic Ram. Where a small stream of water having a fall of a [...] or more flows near the premises, an hydraulic ram may be used to great adva[ntage]

ıter for domestic purposes, or even for irrigation. The ram : momentum of the water flowing through the drive-pipe and ɔ an open tank. Water can be conveyed by a ram 13 000 ft ı ft, provided there is sufficient fall. The drive-pipe supplying : 30 or 40 ft long to give the necessary momentum. The use most economical method of pumping water, as there is no enance except for repairs, and the cost of installation, also, is

ı **of the Rife Rams** are given in the following table. The rmined from the table by multiplying the available supply of ɔr the rated amount of water a Rife ram will use, by the factor at the intersection of the line giving the fall available, for the column showing the height the water is to be elevated. The ll and 50-ft discharge is 192, and this multiplied by the supply of water per minute will give the delivery per day. This is shown by the example worked out in the corner of the table. These capacities are based on efficiencies dependent on the ratio of fall to lift. A fall of 10 ft and a lift of 50 ft give a ratio of 1 to 5, and an efficiency of 66⅔%. The efficiencies of Rife rams based on various ratios, are also given in the table.

Deep Wells and Plunger-Pumps. The common method of obtaining a private water-supply is to drive a deep well until a sufficient supply of water is obtained. The depth to which a well must be driven will, of course, depend upon the locality, and can only be determined by drillings. As the well is driven, a large wrought-iron pipe is sunk to form the casing. Casings are seldom less than 6 or more han 10 in inside diameter, 8 n being the common size. When the water-pocket has een reached, the water will usually rise and stand in the ipe several hundred feet above ts bottom, and the amount of ater that can usually be umped from such wells, without lowering the water, is practically unlimited. The cost f drilling deep wells, per foot THE CASING, differs, of course, with the strata, location and s. As a rule, however, it will average about $5 per foot for a rock and $6 per foot for a well through sand. For raising ɔpen tank a single-acting pump consisting of a working-ı operates a cylinder placed in a smaller pipe lowered into hich the water is raised, is commonly employed. The erably be placed below the water-line in the well, and is th the working-head by wooden sucker-rods. The working-

Fig. 2. Deep-well Working-head for Belt-attachment

Table of Water Required for Rife Rams

Number	Dimensions: Height, ft in	Dimensions: Length, ft in	Dimensions: Width, ft in	Size of drive-pipe, in	Size of delivery-pipe, in	Gallons per minute required to operate engine, gal	Least no. of feet of fall recommended, ft	Weight, lb
10	2 1	3 2	1 8	1¼	¾	3 to 6	3	150
15	2 1	3 4	1 8	1½	¾	5 to 12	3	175
*20	2 3	3 8	1 9	2	1	10 to 18	2	225
25	2 3	3 9	1 9	2½	1	11 to 24	2	250
30	2 7	3 10	1 10	3	1¼	15 to 35	2	275
40	3 3	4 4	2 0	4	2	30 to 75	2	600
80	7 4	8 4	2 8	8	4	150 to 350	2	2 500
*120	......	......	......	12	5	375 to 750	2	3 000
†120	8 9	9 6	3 8	12 (two)	6	750 to 1 500	2	5 500

* Single. † Duplex.

Table of Capacities of Rife Rams

Power-head or fall in ft	4	10	15	20	30	40	*50	60	70	80	90	100	120	140	160	180	[illegible]
2	540	192	128	96	64	43	29	24	....	....	....	....	....	....	....	....	....
3	...	301	192	144	96	72	58	43	37	27	24	....	....	....	....	....	...
4	...	432	256	192	128	96	77	64	55	43	38	29	24	....	....	....	....
5	...	540	345	240	160	120	96	80	69	60	53	43	30	26	....	....	....
6	...	...	432	302	192	144	115	96	82	72	64	57	43	31	27	24	...
7	...	...	505	378	235	168	134	112	96	84	75	67	50	36	31	28	[illegible]
8	...	...	...	432	270	192	154	128	110	96	86	77	64	55	43	38	[illegible]
9	...	...	...	485	300	216	173	144	124	108	96	86	72	62	54	43	[illegible]
*10	...	...	...	540	360	252	*192	160	137	120	107	96	80	68	60	53	[illegible]
12	...	...	...	...	430	301	230	192	165	144	128	115	96	82	72	64	[illegible]
14	...	...	...	...	505	353	270	224	192	168	150	135	112	96	84	75	[illegible]
16						432	323	257	220	192	171	154	128	110	96	85	[illegible]
18						486	390	303	247	216	192	173	144	124	108	96	[illegible]
20						540	430	336	288	240	214	192	160	137	120	107	[illegible]
22						....	475	370	303	264	235	212	176	151	132	118	[illegible]
24						....	520	405	346	288	256	230	192	164	144	128	[illegible]
26						....	....	470	375	328	278	250	208	178	156	139	[illegible]
28						....	....	505	430	354	300	260	224	192	168	149	[illegible]
30						....	....	540	465	405	336	288	240	206	180	160	[illegible]

Example

With a supply of 1 400 gal per min, 10-ft fall, 50-ft elevation, No. 120 engine will deliver 268 800 gal per day.
1 400 × 192 = 268 800

* Multiply factor opposite POWER-HEAD and under PUMPING-HEAD by the number gallons per minute USED by the engine; the result will be the number of gallons DELIVERED per day.

The efficiency developed is governed by the ratio of fall to pumping-head.

The efficiency of rife rams is based on....
- 75% for a ratio of 1 to 2¼
- 70% for a ratio of 1 to 3
- 66⅔% for a ratio up to 1 to 18
- 60% for a ratio up to 1 to 23
- 50% for a ratio up to 1 to 30

ad may be operated by hand, or by a crank-rod attached to a pumping-jack, admill or engine. With a single-acting pump the plunger is raised and tered once with every revolution of the driving-wheel, the principle of oper-on being the same as in an ordinary hand suction-pump. Fig. 2 shows a nple arrangement for operating a working-head by belt-power. This is own as a deep-well power working-head. A DEEP-WELL PUMP (Fig. 2) differs m a SUCTION-PUMP in that it will raise water from any depth, whereas a suc-n-pump in practice will raise water only about 20 ft. A suction-pump may placed at any point in relation to the well, and will draw the water any sonable horizontal distance. The deep-well pump, on the other hand, must set directly over the well, but it will then deliver the water at any desired it. The amount of water pumped in a minute by any single-acting pump etermined by the diameter of the suction-cylinder, the length of stroke, and number of strokes per minute. The table following gives the capacity per ke for cylinders of different diameters, and for strokes of different lengths. ind the capacity per minute, multiply the values given in the table by the lutions per minute. The usual speed of single-acting working-heads and ping-jacks is from 25 to 30 revolutions per minute. Cylinders over 2¾ in in ieter should have a substantial iron working-head.

able Showing Capacity of Single-Acting Pumps of Given Diameter and Length of Stroke

m. n- in es	Length of stroke in inches								
	6	8	10	12	14	16	18	20	24
	Capacity per stroke in gallons								
	0.0319	0.0425	0.0531	0.0637	0.0743	0.0848	0.0955	0.1062	0.1274
	0.0385	0.0513	0.0642	0.0770	0.0890	0.1027	0.1156	0.1280	0.1541
	0.0459	0.0612	0.0765	0.0918	0.1071	0.1224	0.1377	0.1530	0.1836
	0.0625	0.0833	0.1041	0.1249	0.1457	0.1666	0.1874	0.2082	0.2499
	0.0816	0.1088	0.1360	0.1632	0.1904	0.2176	0.2448	0.2720	0.3264
	0.1033	0.1377	0.1721	0.2063	0.2410	0.2754	0.3096	0.3442	0.4128
	0.1275	0.1700	0.2125	0.2550	0.2975	0.3400	0.3825	0.4250	0.5100
	0.1543	0.2057	0.2571	0.3085	0.3598	0.4114	0.4626	0.5142	0.6170
	0.1836	0.2448	0.3060	0.3672	0.4284	0.4896	0.5508	0.6120	0.7344
	0.2154	0.2872	0.3594	0.4312	0.5030	0.5748	0.6466	0.7182	0.8624
	0.2499	0.3332	0.4165	0.4998	0.5831	0.6664	0.7497	0.8330	0.9996
	0.2868	0.3824	0.4780	0.5736	0.6692	0.7648	0.8605	0.9561	1.1470
	0.3264	0.4352	0.5440	0.6528	0.7616	0.8704	0.9792	1.0880	1.3056
	0.3684	0.4912	0.6141	0.7368	0.8596	0.9824	1.1050	1.2280	1.4730
	0.4131	0.5508	0.6885	0.8262	0.9639	1.1016	1.2393	1.3770	1.6524
	0.4602	0.6136	0.7671	0.9204	1.0730	1.2270	1.3800	1.5340	1.8400

Air Engines. These are very extensively used for pumping water for houses, as they are absolutely safe, require little attention, and have es, springs or gauges to get out of order. They are also adapted to al-ly kind of fuel, such as coal, coke, wood, gas, or kerosene oil. They ıp from either a shallow or a deep well, but are best adapted to wells

in which the surface of the water is within 20 ft of the top of the well. Th best known hot-air engines are the Rider-Ericsson, which have been in succes ful operation for many years. These engines have capacities ranging from 1 to 3 500 gal per hour and will deliver water from 50 to 350 ft above t surface of water in the well, although the higher the water is raised the l will be the quantity delivered. The cost of these engines, with pump a tached, varies from $110 for the smallest size, having a capacity of 150 gal p hour raised 50 ft, to $540 for the largest size, having a capacity of 3 500 gal p hour raised 50 ft. The smaller size requires about 1 quart of kerosene or 3 of anthracite coal per hour. Hot-air engines should be placed close to t source of supply, and when the latter is a deep well the engine must placed so that the pump-rod will be in a vertical line above the cylinder the well, the operation of pumping being the same as that of the ordina single-acting deep-well pump. It is not practicable to DRAW water more th from 20 to 25 ft, in height, with any form of suction-pump, because of t difficulty of keeping the pipe, valve and fittings absolutely air-tight. F further information, see the catalogue of the Rider-Ericsson Engine Compan

Action of Wind and Capacities of Pumping Windmills

Velocity per hour in miles	Pressure* per square foot in pounds	Description of wind	Action of wind and windmill
3	0.045	Just perceptible.......	Windmills will not run
5	0.125	Pleasant wind.........	Might start if lightly loaded
8	0.33	Fresh breeze..........	Will start pumping
10	0.5	Average wind.........	Pumps nicely if properly load
15	1.125	Good working wind...	Does excellent work
20	2	Strong wind..........	Gives best service
25	3.125	Very strong wind......	Maximum results secured
30	4.5	Gale................	Should be furled out of wind
40	8	Storm................	Well-constructed mills an towers safe if properly erect
50	12.5	Severe storm..........	
60	18	Violent storm.........	Buildings, trees, etc., might injured
80	32	Hurricane............	Buildings, trees, etc., woul be injured
100	50	Tornado	Ruin

From the above table it will be seen that the only available winds are those blo with a velocity of from 8 to 25 miles per hour, and that a 15-mile wind can be util to the best advantage. It is therefore advisable to LOAD a windmill for a 15-mile w It then starts pumping in an 8-mile wind, does excellent work in a 15-mile wind reaches the maximum results in a 25-mile wind.

* The pressures per square foot in pounds will vary slightly from the values g according to the formula which is used to obtain such pressures. See, also, Cha XXVII, pages 1052-3, Chapter XXX, page 1199, and page 1717.

Windmills. In the country and on large suburban estates, windmills extensively used for pumping water. Aside from the noise of operation, only objection to the windmill, where it can be used, is the irregularity o supply, but with a large storage-tank this is not a serious objection when for domestic purposes only. Professor Thurston says, regarding windm "In estimating the capacity, a working-day of eight hours is assumed, but

iachine, when used for pumping, may actually do its work twenty-four hours a ay for days, weeks, and even months together, whenever the wind is stiff enough) turn it. It costs for work done only one-half or one-third as much as steam, ot-air, or gas-engines of similar power." The action of wind of different elocities, the pressure per square foot of sail-surface and its relation to the umping capacity of pumps can be found in the following table, compiled by airbanks, Morse & Company.

The windmill operates the plunger in the well, the process of pumping being e same as that of the single-acting pumps described above. The following ble of capacity was prepared by Alfred R. Wolff, and is sufficiently accurate r all practical purposes:

Capacity of the Windmill

Desig-ation of mill wheel, ft	Velocity of wind in miles per hour	Revolutions of wheel per minute	Gallons of water raised per minute to an elevation of						Equivalent actual useful h.p. developed
			25 ft	50 ft	75 ft	100 ft	150 ft	200 ft	
8½	16	40 to 50	6.192	3.016	....	....	....	....	0.04
10	16	35 to 40	19.179	9.563	6.638	4.750	....	....	0.12
12	16	30 to 35	33.941	17.952	11.851	8.435	5.680	....	0.21
14	16	28 to 35	45.139	22.569	15.304	11.246	7.807	4.998	0.28
16	16	25 to 30	64.600	31.654	19.542	16.150	9 771	8.075	0.41
18	16	22 to 25	97.682	52.165	32.513	24.421	17.485	12.211	0.61
20	16	20 to 22	124.950	63.750	40.800	31.248	19.284	15.938	0.78
25	16	16 to 18	212.381	106.964	71.604	49.725	37.349	26.741	1.34

The horse-power of windmills of the best construction is proportional to the ares of their diameters and inversely as their velocities; for example, a 10-ft in a 16-mile breeze will develop 0.15 horse-power at 65 revolutions per min-; and with the same breeze:

a 20-ft mill, at 40 revolutions per minute, 1 horse-power;
a 25-ft mill, at 35 revolutions per minute, 1¾ horse-power;
a 30-ft mill, at 28 revolutions per minute, 3½ horse-power.

he wheels of very few windmills are larger than 25 ft in diameter. There no pumps which will enable the user of a windmill to utilize the increased er obtained from winds of high velocity, so that in practice the amount of er pumped by windmills in high winds is but little more than is pumped by same mills in winds having velocities of from 12 to 18 miles per hour. For reason it is customary to regulate windmills to govern at about 25 miles an . Theoretically the increase in power from increased velocity of wind is equal ne square of its proportional velocity; as, for example, the 25-ft mill rated e for a 16-mile wind will, with a 32-mile wind, have its horse-power increased × 1¾ = 7 horse-power. A windmill "will run and produce work in an 8- breeze." Windmills have also been used for the generating and storage ectricity for small lighting-plants.*

r-Lift Process. Compressed air is now being used to an increasing extent aising water from artesian wells. The process in general consists of sub- ing a discharge-pipe in a closed well, with a smaller pipe inside delivering

* See Kent's Mechanical Engineers' Pocket-Book.

compressed air into it at the bottom. The compressed air by its inherent e pansive force lifts a column of mingled air and water which is conveyed to a open tank, to permit of the escape of the air. If desired the water may then conveyed by gravity into a series of closed tanks, and forced by air-pressure different parts of a building, the only machinery required being an air-co pressor and power for driving it The slip of the bubble constitutes the ch loss of energy in the air-lift. The method of piping a well differs according its general conditions and the quantity of water to be pumped. "No two we are alike, and consequently the method of piping which might be applied one would be unsuited to another." Information as to the best method piping any particular well may be obtained from the Ingersoll-Sergeant Dr Company.

Advantages of the Air-Lift Process. From two to six times as much wa may be obtained from a given diameter of well as with any other known syste because there are no valves, cylinders, or rods to hinder the rapid discharge water. One air-compressor operates any number of wells, which may be a distance apart so as not to affect one another. There is nothing outside t engine-room to look after or wear out. Nothing but common pipe in the wel Sand or gravel does no harm. The COST of raising 1 000 gal of water by t method, including fuel, labor, oil, interest on cost of well, boiler, compress foundations, pipes, real estate, erection and taxes, including 15% for dep ciation, runs from 2½ cts down to ¼ ct, according to the size of the pla height of lift, and other local conditions. With the average outfit of mediu or small size, it is usually under 1¼ cts.* The air-lift process is now exte sively used in ice-works, breweries, cold-storage houses, textile mills, dye-wor etc., and a great variety of industrial plants, and for the water-supply quite a number of the smaller cities. In Newark, N. J., pumps of this ty are at work having a total capacity of 1 000 000 gal daily, lifting water fro three 8-in artesian wells.†

Pneumatic Water-Supply Systems. The pneumatic system of supplyi water to buildings is used extensively in buildings and institutions remote fro public water-supplies. With the pneumatic system, instead of an open el vated tank, a closed water-tight tank of iron or steel is used, and this tank m be located at any level, for the water is forced from it by means of compress air confined in the top of the tank. This fact makes it possible to bury the ta in the ground below the frost-line, away from the heat of the sun, and where t water will have an almost uniform temperature the year round. The water protected from possible contamination from insects, rats, birds, dust, or oth agencies, while the tank takes up no valuable space above ground, imposes weight upon the attic-floor of a building, and does not disfigure the landsca The principle of operation is this: Air is compressible, while water is not. then, water is pumped into a closed tank at the bottom, it will trap the a within, and the more water pumped in, the greater the compression of the a The elasticity of the air, then, will force the water out again, whenever a fauce opened, and the water will continue to flow as long as the air is under sufficient pr sure in the tank. In practice the air would become absorbed by the water in t tank, and in a short time become exhausted, if it were not supplied as fast as us This is accomplished by injecting a proportionate amount of air with each stro of the pump, by means of a SNIFTER-VALVE air-compressor, or other device. connections to the tank are taken from the bottom, to prevent the escape of which would occur if the connections were taken from the top of the tank.

*. Ingersoll-Rand Drill Company, St Louis, Mo. † Kent.

Horse-Power Required to Raise Water to Different Heights

General Principles. The power required to raise a certain quantity of water to certain height varies directly with the quantity to be raised, and also with the eight. For instance, it requires twice as much power to raise 200 gal per minute) ft high as it does to raise 100 gal to the same height and in the same time; and to ise 100 gal 20 ft high requires twice as much power as it does to raise 100 gal 10 high. To find the theoretical horse-power necessary to elevate water to a given ight, multiply the number of gallons per minute by 8.335, the weight of 1 gal, d this result by the total number of feet the water is raised, that is, from the rface of the water to the highest point to which the water is raised, and the sult gives the power in foot-pounds; divide by 33 000, and the quotient is e horse-power. To the theoretical power a liberal allowance must be made r the inefficiency of the pump. For a cylinder-pump add from 75 to 100%. the actual height to which the water is to be raised add the friction-loss in t, given in Table F, page 1388, when the discharge is to be piped any distance.

Example. Find the theoretical horse-power required to raise 100 gal per nute 120 ft high, through a 3-in pipe, 200 ft long.

Solution. From Table F, the friction-head for 100 gal per min in a 3-in pipe,) ft long, is 1.31 × 2.3 or 3 ft. For 200 ft it will be 6 ft, which, added to 120, es 126 ft for the height. Then theoretical horse-power = 100 × 8.35 × 126/ 000 = 3.2 h.p. The actual horse-power required will probably vary from o 6, according to the efficiency of the pump. The mistake of using too ill a discharge-pipe can easily be seen from Table F. For instance, if attempted to force 100 gal per minute through 100 ft of 2-in pipe, back-pressure would be equivalent to raising the water 22 ft high. The used would be correspondingly increased. Right-angle turns are to be ided, as the friction is very materially increased, being practically equal to friction of 25 ft of straight pipe.

Table of Effective Fire-Streams

Jsing 100 ft of 2½-in ordinary best-quality rubber-lined hose between nozzle and hydrant or pump

Smooth nozzle	¾ in					⅞ in				
essure at hydrant, lb	32	54	65	75	86	34	57	69	80	91
essure at nozzle, lb	30	50	60	70	80	30	50	60	70	80
rtical height, ft	48	67	72	76	79	49	71	77	81	85
rizontal distance, ft	37	50	54	68	62	42	55	61	66	70
l discharged per min	90	116	127	137	147	123	159	174	188	201
Smooth nozzle	**1 in**					**1⅛ in**				
ssure at hydrant, lb	37	62	75	87	100	42	70	84	98	112
ssure at nozzle, lb	30	50	60	70	80	30	50	60	70	80
tical height, ft	51	73	79	85	89	52	75	83	88	92
izontal distance, ft	47	61	67	72	76	50	66	72	77	81
discharged per min	161	208	228	246	263	206	266	291	314	336

e—Streams. The following is an extract from a paper read by John reeman at a meeting of the New England Waterworks Association, ed Some Experiments and Practical Tables Relating to Fire-Streams,

"When unlined linen hose is used the friction or pressure-loss is from 8 t
60%, increasing with the pressure. This kind of hose is best for inside use i
short lengths. Mill-hose is better than unlined linen hose for long length
but ordinarily the best quality of smooth rubber-lined hose is superior to th
mill-hose, having less frictional resistance. The ring-nozzle is inferior to th
smooth nozzle and actually delivers less water than the smooth nozzle. Fo
instance, the 7/8-in ring-nozzle discharges the same quantity of water as
3/4-in smooth nozzle, and a 1-in ring-nozzle the same as a 7/8-in smooth nozzl
Two hundred and fifty gallons per minute is a good standard fire-stream a
80-lb pressure at the hydrant; 100-lb pressure should not be exceeded excep
for very high buildings or lengths of hose exceeding 300 ft."

Notes on the Construction of Cylindrical Wooden Tanks*

Material should be either cedar, cypress, juniper, fir, yellow pine, or whit
pine, free from imperfections and thoroughly air-dry. Clear Louisiana re
Gulf cypress makes the most durable tanks.

Staves and Bottom of tanks of greater capacities than 15 000 gal should b
made of 2½-in, dressed to about 2¼ in, stock for tanks 12 ft and not exceedin
16 ft diameter or 16 ft deep. For larger tanks 3-in, dressed to about 2¾ i
stock should be used. For smaller tanks 2-in stock may be used. Stave
should be connected about one-third the distance from the top by a ⅝-i
dowel to hold them in position during erection. The bottom planks should b
dressed on four sides, and the edges of each plank should be bored with hol
not over 3 ft apart for ⅝-in dowels.

Taper. The batter to each side should not be less than ¼ in nor more tha
½ in per ft.

Hoops should be of ROUND wrought iron or mild steel of good quality. Wrough
iron is preferable because it does not rust as easily as steel. There should be n
welds in any of the hoops. Where more than one length of iron is necessar
lugs should be used to make the joints; and when more than one piece is necessa
the several pieces constituting one hoop should be tied together in preparing f
shipment. Hoops for fire-tanks should be of such size and spacing that the stre
in no hoop will exceed 12 500 lb per sq in when computed from the area at ro
of thread. For general purposes, a stress of 15 000 lb per sq in is permissibl
On acccount of the swelling of the bottom planks, the hoops near the botto
may be subjected to a stress greater than that due to the water-pressure alon
additional hoops, therefore, should be provided. For tanks up to 20 ft in diam
eter, one hoop of the size used next above it should be placed around the botto
opposite the croze and not counted upon as withstanding any water-pressur
For tanks 20 ft or more in diameter, two hoops, as above, should be use
Hoops with UPSET ends must not be used. The top hoop should be plac
within 2 in of the top of staves, so that the overflow-pipe may be inserted as hi
as possible. Hoops should be so placed that the lugs will not be in a vertic
line. No hoop should be less than ¾ in in diameter. All should be clean
of mill-scale and rust and painted one coat of red lead, lampblack and boiled
before erecting.

Note. The strength of a tank depends chiefly on its hoops. Round hoo
are specified because they do not rust rapidly; a slight amount of rust do

* These notes have been condensed from specifications published by the Inspecti
Department of the Factory Mutual Fire Insurance Company, 31 Milk Street, Boston;
most excellent pamphlet.

not have the same weakening effect as on a flat hoop, and round hoops are not likely to burst when the tank swells, as they will sink into the wood.

Spacing of Hoops. The hoops should be spaced so that each one will have the same stress per square inch, and no space should be greater than 21 in. To meet this requirement the hoops must be spaced quite close together at the bottom, the space between them gradually increasing towards the top.

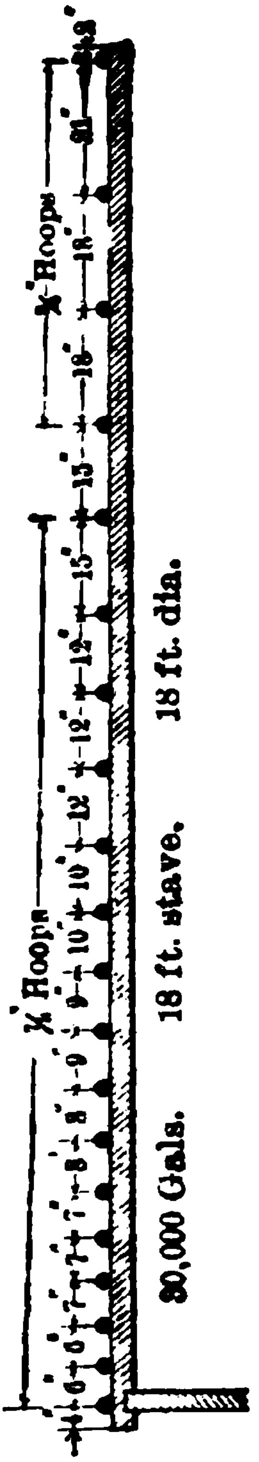


Fig. 4. Lug for Tank-hoops

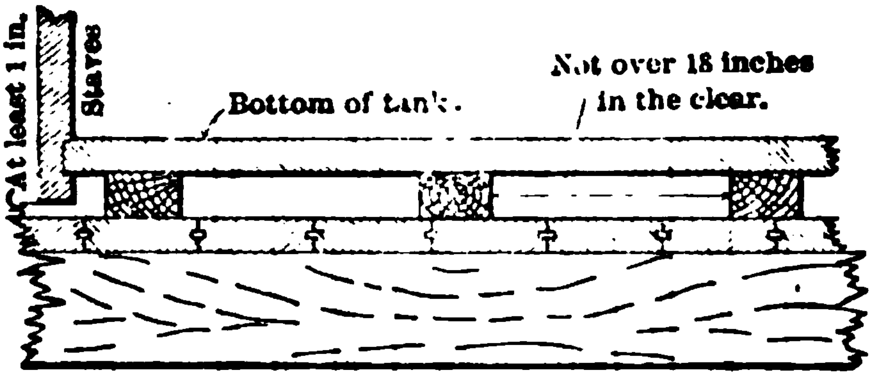


Fig. 5. Support for Bottom of Tank

Fig. 3 shows the proper spacing of hoops for a tank 18 ft in diameter, with 18-ft staves. The spacing for seven other sizes of tanks is given in the pamphlet referred to. It may be computed by the following formula:

$$\text{Spacing of hoops in inches} = \frac{\text{strength}}{2.6 \times \text{diameter in feet} \times H}$$

For strength of a ¾-in rod use 3 750; of a ⅞-in rod, 5 250; of a 1-in rod, 6 875; and of a 1⅛-in rod, 8 625.

H is the distance from surface of the water to center of hoop in feet.

Example. How far apart should 1-in hoops be placed, at 15 ft 2 in from top of tank, on a tank 20 ft diameter?

Solution. $$\text{Spacing} = \frac{6\,875}{2.6 \times 20 \times 15} = 8\tfrac{3}{4}\text{ in}$$

. 3. Diagram
f Hoop-spacing
r Tanks

Lugs should be as strong as the hoops. A lug similar to Fig. 4 is simple and fulfils the requirement for strength. Malleable lugs are required.

Support. The weight of the tank should be supported entirely from its tom; and in no event should any weight come on the bottom of the staves. e planks upon which the tank-bottom rests should cover at least one-fifth area of the bottom, should be not over 18 in apart, and of such thickness t the bottom of the staves will be at least 1 in from the floor (see Fig. 5).

he Discharge-Pipe should preferably leave the bottom of the tank at its ter and extend up inside of the tank 4 in, to allow the sediment to collect he bottom of the tank.

The Overflow-Pipe should be placed as near the top of the tank as possible, discharging either through side or bottom, as may be desired. An overflow is much to be preferred to a telltale, as the latter is liable to get out of order.

Heating. Tanks of moderate size need to be provided with some means to prevent freezing. When a tank is in an enclosed room, as in a mill-tower, the best method is to keep the room warm by a coil of steam-pipe with a return to the boiler-room. A covered tank out of doors may often be similarly heated by placing the steam-pipe in the bottom of the tank. With a tank located on a high trestle, or at a distance from the steam-supply, it is often impracticable to arrange a return-pipe. In this case steam may be blown directly into the water in the tank. A 1-in pipe is generally sufficient for this purpose. It should be carried to the top of the tank and there bend over and dip downwards, so that its outlet is about 1 ft below the high-water line. A check-valve should be placed in this steam-pipe, near its point of discharge, to prevent water being drawn back by siphon-action when the steam is shut off. The water in fire-tanks must be kept from freezing by means of a water-heater which either heats a coil in the tank, or circulates a current of water through the tank.

⅞ in. horizontal nailing strips

⅞ Thick ... ad around each box except outside, grooved sheathing.

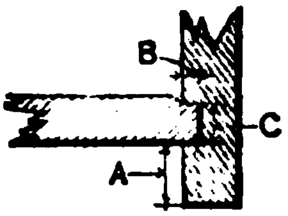
Fig. 6. Method of Frostproofing Pipes

Frostproofing for Pipes. The discharge-pipe from a tank on a trestle, or from one elevated above a roof, must be protected from freezing. The common practice is to enclose the pipe in a double, triple, or quadruple box made of boards and tarred paper, as shown in Fig. 6. If steam is supplied to the tank, the steam-pipe is carried inside the box. In New England, New York State and Canada the quadruple boxing is generally used, whereas in the milder regions to the south triple or double boxing is used. The boxing should always be carried down into the ground below the frost-line, and a good tight joint made at the underside of the tank.

Covers. For economy in heating and to prevent birds, leaves, etc., from getting into the water, all out-of-door tanks should be covered. A double cover is recommended consisting of a tight flat cover made of matched boards supported by joists which span the top of the tank, and above this a shingled conical roof. To prevent the covering from being blown off, it should be firmly fastened to the top of the tank by straps of iron. In order to keep out the wind particular attention should be given to making a tight joint where the roof rests on the top of the staves.

Scuttles should be arranged in both the conical and flat covers to give access to the inside of the tank and a substantial, permanent ladder erected to give easy access to the top of the tank.

Dimensions of Tanks of Standard Sizes

Approximate net capacity, gal	Size. Outside dimensions: Average diameter, ft	in	Length of stave, ft	Thickness of lumber after being machined: Staves, in	Bottom, in	 A, in	B, in	C, in	Hoops: Number of	Size, in
10 000	13	4	12	2¼	2¼	3½	⅝	2⅛	11	¾
15 000	14	6	14	2¼	2¼	3½	⅝	2⅛	14	¾
20 000	15	6	16	2¼	2¼	3½	⅝	2⅛	5 11	¾ ⅞
25 000	17	6	16	2¾	2¾	3½	¾	2⅝	4 12	¾ ⅞
30 000	18	0	18	2¾	2¾	3½	¾	2⅝	4 16	¾ ⅞
50 000	22	0	20	2¾	2¾	3½	¾	2⅝	4 19	⅞ ⅞
75 000	24	6	24	2¾	2¾	3½	¾	2⅝	4 6 21	⅞ 1 1⅛
100 000	28	6	24	2¾	2¾	3½	¾	2⅝	5 29	1 1⅛

Pumps for Fire-Streams. The dimensions of steam-pumps for fire-protection in buildings, approved by the Board of Underwriters, can be found in the following table.

Underwriter Steam Fire-Pumps

Rated capacity, gal per min	Size in inches: Diameter of steam-piston, in	Diameter of water-plunger, in	Length of stroke, in	Boiler h.p. required, A.S.M.E. standard, h.p.	Size in inches: Suction-pipe, in	Discharge-pipe, in	Steam supply-pipe, in	Steam exhaust-pipe, in	Relief-valve, in	Piston-rod, in	Valve-rod, in	Over-all dimensions of largest pump of given capacity: Length, ft	in	Width, ft	in	Height, ft	in	Proper capacity for priming-tank, gal
500	14 14 16	7 7¼ 8	12 12 10	80	8	6	3	4	3	2	1	9	⅛	5	2	7	5	250
750	16 16	9 9¼	12 12	115	10	7	3½	4	3½	2¼	1⅛	9	5	5	2	8	0	375
1000	18 18½	10 10½	12 10	150	12	8	4	5	4	2⅜	1⅛	10	8	5	7½	8	10	500
1500	20	12	16	200	14	10	5	6	5	2½	1¼	12	5	5	7	8	11	750

The capacities given in last column are desirable; but in case the suction-pipe is short and the lift low, a tank of not less than one-half the capacity stated may sometimes be used.

Notes on Steel Tanks *

Steel Tanks of sizes commonly used for fire-protection cost from 40 to 100% more than wooden tanks. The additional cost for large tanks is relatively less than for small tanks. A steel tank of about 40 000-gal capacity or over can be erected on a steel trestle at about the same cost as a wooden tank, since a saving can be made in the cost of supports by making a hemispherical or conical bottom to the steel tank and supporting the tank directly on the legs of the trestle, thus saving the expense of horizontal supporting beams. A steel tank is superior to a wooden tank for the following reasons: (1) It will last for an indefinite time if KEPT THOROUGHLY PAINTED inside and out, whereas a wooden tank will have to be replaced in from twelve to thirty years, usually in about fifteen years; (2) it will be absolutely tight when once well erected and properly cared for, whereas a wooden tank will shrink and leak if the water gets low; (3) it will not be at all likely to burst suddenly, if originally correctly designed, even if painting is neglected, for experience shows that a few spots will first rust through and thus show the weak condition by small leaks, whereas a wooden tank, if neglected, may burst its hoops suddenly and cause serious damage. The objections to steel tanks are that: (1) They require skilled boiler-makers to erect them, thus adding considerable to the cost when erected at a distance from a boiler-shop; (2) they are more difficult to protect against freezing; (3) they give more trouble by SWEATING when placed in a mill-tower; (4) they deteriorate rapidly if painting is neglected.

Stresses in Cylindrical Tanks.† The intensities of stresses in lb per sq in found in cylindrical tanks are as follows: A tensile stress due to hydrostatic pressure at any vertical joint or section of the shell of a tank filled with water,

$$S = 62.5\,HD/(2 \times 12\,t) = 2.6\,HD/t$$

A compressive stress at any horizontal joint or section, due to the weight of the stack,

$$S = W/(\pi D \times 12\,t) = 0.026\,W/Dt$$

A stress at any horizontal joint or section, tensile on the windward side and compressive on the leeward side, due to the wind, $S_2 = 0.106\,M/D^2t$. (See Self-Sustaining Steel Chimneys, page 1376.) In the above equation, H = height of tank in ft above section considered, D = diameter in ft, t = thickness of shell in in, W = weight of tank in lb, and M = bending moment in ft-lb. The conditions for overturning from wind are most severe when the tank is empty.

Stand-Pipes were much used for storage-reservoirs at one time. They usually varied from 12 to 30 ft in diameter and from 35 to 120 ft in height. A tank built in 1889 at Greenwich, Conn., was 80 ft in diameter and 30 ft in height. Its capacity was 1 300 000 gal. A stand-pipe built in 1876 at Winona, Minn, was 4 ft in diameter by 210 ft in height. The steel cylinder was surrounded by a masonry tower. A long list of failures, mostly due to faulty design, are recorded against the stand-pipe. Because of this and the superior advantage of the elevated water-tower, few are now built. General Specifications for Elevated Steel Tanks on Towers, and for Stand-pipes (Trans. Am. Soc. C. E., Vol. 64, 1909, pages 548 to 566), and General Specifications for Steel, Water and Oil-Tanks (Proc. Am. Ry. Eng. Asso., vol. 13, 1912), are both reprinted in Ketchum's Structural Engineers' Handbook.

* Inspection Department of the Factory Mutual Insurance Company, Boston.
† From Notes by Robins Fleming.

n Cubic Feet and U. S. Gallons of Pipes and Cylinders of Various Diameters and One Foot in Length

1 gallon = 231 cu in. 1 cu ft = 7.4805 gal

For 1 ft in length		Diameter in inches	For 1 ft in length		Diameter in inches	For 1 ft in length	
u ft, lso a in q ft	U. S. gal 231 cu in		Cu ft, also area in sq ft	U. S. gal, 231 cu in		Cu ft, also area in sq ft	U. S. gal, 231 cu in
.0003	0.0025	6¾	0.2485	1.859	19	1.969	14.73
.0005	0.0040	7	0.2673	1.999	19½	2.074	15.51
.0008	0.0057	7¼	0.2867	2.145	20	2.182	16.32
.0010	0.0078	7½	0.3068	2.295	20½	2.292	17.15
.0014	0.0102	7¾	0.3276	2.450	21	2.405	17.99
.0017	0.0129	8	0.3491	2.611	21½	2.521	18.86
.0021	0.0159	8¼	0.3712	2.777	22	2.640	19.75
.0026	0.0193	8½	0.3941	2.948	22½	2.761	20.66
0.0031	0.0230	8¾	0.4176	3.125	23	2.885	21.58
0.0036	0.0269	9	0.4418	3.305	23½	3.012	22.53
0.0042	0.0312	9¼	0.4667	3.491	24	3.142	23.50
0.0048	0.0359	9½	0.4022	3.682	25	3.409	25.50
0.0055	0.0408	9¾	0.5185	3.879	26	3.687	27.58
0.0085	0.0638	10	0.5454	4.080	27	3.976	29.74
0.0123	0.0918	10¼	0.5730	4.286	28	4.276	31.99
0.0167	0.1249	10½	0.6013	4.498	29	4.587	34.31
0.0218	0.1632	10¾	0.6303	4.715	30	4.909	36.72
0.0276	0.2066	11	0.6600	4.937	31	5.241	39.21
0.0341	0.2550	11¼	0.6903	5.164	32	5.585	41.78
0.0412	0.3085	11½	0.7213	5.396	33	5.940	44.43
0.0491	0.3672	11¾	0.7530	5.633	34	6.305	47.16
0.0576	0.4309	12	0.7854	5.875	35	6.681	49.98
0.0668	0.4998	12½	0.8522	6.375	36	7.069	52.88
0.0767	0.5738	13	0.9218	6.895	37	7.467	55.86
0.0873	*0.6528	13½	0.9940	7.436	38	7.876	58.92
0.0985	0.7369	14	1.0690	7.997	39	8.296	62.06
0.1134	0.8263	14½	1.1470	8.578	40	8.727	65.28
0.1231	0.9206	15	1.2270	9.180	41	9.168	68.58
0.1364	1.0200	15½	1.3100	9.801	42	9.621	71.97
0.1503	1.1250	16	1.3960	10.440	43	10.085	75.44
0.1650	1.2340	16½	1.4850	11.110	44	10.559	78.99
0.1803	1.3490	17	1.5760	11.790	45	11.045	82.62
0.1963	1.4690	17½	1.6700	12.490	46	11.541	86.33
0.2131	1.5940	18	1.7680	13.220	47	12.048	90.13
0.2304	1.7240	18½	1.8670	13.960	48	12.566	94.00

* Actual.

nd the capacity of pipes greater than those given, look in the table for a one-half the given size and multiply its capacity by 4, or one of one-s size and multiply its capacity by 9, etc. To find the WEIGHT of water of the given sizes, multiply the capacity in cubic feet by the weight of a oot of water at the temperature of the water in the pipe.

nd the capacity of a cylinder in U. S. gallons, multiply the length by the of the diameter and by 0.0034.

Cylindrical Vessels, Tanks, Cisterns, Etc.

Diameter in feet and inches, area in square feet, and U. S. gallons capacity for 1 ft in depth

1 gallon = 231 cu in = 0.1337 cu ft

Diam, ft	Diam, in	Area, sq ft*	Gal, 1-ft depth	Diam, ft	Diam, in	Area, sq ft*	Gal, 1-ft depth	Diam, ft	Diam, in	Area, sq ft*	Gal, 1-ft depth
1		0.785	5.87	5	8	25.22	188.66	19		283.53	2120.9
1	1	0.922	6.89	5	9	25.97	194.25	19	3	291.04	2177.1
1	2	1.069	8.00	5	10	26.73	199.92	19	6	298.65	2234.0
1	3	1.227	9.18	5	11	27.49	205.67	19	9	306.35	2291.7
1	4	1.396	10.44	6		28.27	211.51	20		314.16	2350.1
1	5	1.576	11.79	6	3	30.68	229.50	20	3	322.06	2409.2
1	6	1.767	13.22	6	6	33.18	248.23	20	6	330.06	2469.1
1	7	1.969	14.73	6	9	35.78	267.69	20	9	338.16	2529.6
1	8	2.182	16.32	7		38.48	287.88	21		346.36	2591.0
1	9	2.405	17.99	7	3	41.28	308.81	21	3	354.66	2653.0
1	10	2.640	19.75	7	6	44.18	330.48	21	6	363.05	2715.8
1	11	2.885	21.58	7	9	47.17	352.88	21	9	371.54	2779.3
2		3.142	23.50	8		50.27	376.01	22		380.13	2843.6
2	1	3.409	25.50	8	3	53.46	399.88	22	3	388.82	2908.6
2	2	3.687	27.58	8	6	56.75	424.48	22	6	397.61	2974.3
2	3	3.976	29.74	8	9	60.13	449.82	22	9	406.49	3040.8
2	4	4.276	31.99	9		63.62	475.89	23		415.48	3108.0
2	5	4.587	34.31	9	3	67.20	502.70	23	3	424.56	3175.9
2	6	4.909	36.72	9	6	70.88	530.24	23	6	433.74	3244.6
2	7	5.241	39.21	9	9	74.66	558.51	23	9	443.01	3314.0
2	8	5.585	41.78	10		78.54	587.52	24		452.39	3384.1
2	9	5.940	44.43	10	3	82.52	617.26	24	3	461.86	3455.0
2	10	6.305	47.16	10	6	86.59	647.74	24	6	471.44	3526.6
2	11	6.681	49.98	10	9	90.76	678.95	24	9	481.11	3598.9
3		7.069	52.88	11		95.03	710.90	25		490.87	3672.0
3	1	7.467	55.86	11	3	99.40	743.58	25	3	500.74	3745.8
3	2	7.876	58.92	11	6	103.87	776.99	25	6	510.71	3820.3
3	3	8.296	62.06	11	9	108.43	811.14	25	9	520.77	3895.6
3	4	8.727	65.28	12		113.10	846.03	26		530.93	3971.6
3	5	9.168	68.58	12	3	117.86	881.65	26	3	541.19	4048.4
3	6	9.621	71.97	12	6	122.72	918.00	26	6	551.55	4125.9
3	7	10.085	75.44	12	9	127.68	955.09	26	9	562.00	4204.1
3	8	10.559	78.99	13		132.73	992.01	27		572.56	4283.0
3	9	11.045	82.62	13	3	137.89	1031.5	27	3	583.21	4362.7
3	10	11.541	86.33	13	6	143.14	1070.8	27	6	593.96	4443.1
3	11	12.048	90.13	13	9	148.49	1110.8	27	9	604.81	4524.3
4		12.566	94.00	14		153.94	1151.5	28		615.75	4606.2
4	1	13.095	97.96	14	3	159.48	1193.0	28	3	626.80	4688.8
4	2	13.635	102.00	14	6	165.13	1235.3	28	6	637.94	4772.1
4	3	14.186	106.12	14	9	170.87	1278.2	28	9	649.18	4856.2
4	4	14.748	110.32	15		176.71	1321.9	29		660.52	4941.0
4	5	15.321	114.61	15	3	182.65	1366.4	29	3	671.96	5026.6
4	6	15.90	118.97	15	6	188.69	1411.5	29	6	683.49	5112.9
4	7	16.50	123.42	15	9	194.83	1457.4	29	9	695.13	5199.9
4	8	17.10	127.95	16		201.06	1504.1	30		706.86	5287.7
4	9	17.72	132.56	16	3	207.39	1551.4	30	3	718.69	5376.1
4	10	18.35	137.25	16	6	213.82	1599.5	30	6	730.62	5465.4
4	11	18.99	142.02	16	9	220.35	1648.4	30	9	742.64	5555.4
5		19.63	146.88	17		226.98	1697.9	31		754.77	5646.1
5	1	20.29	151.82	17	3	233.71	1748.2	31	3	766.99	5737.5
5	2	20.97	156.83	17	6	240.53	1799.3	31	6	779.31	5829.7
5	3	21.65	161.93	17	9	247.45	1851.1	31	9	791.73	5922.6
5	4	22.34	167.12	18		254.47	1903.6	32		804.25	6016.[illegible]
5	5	23.04	172.38	18	3	261.59	1956.8	32	3	816.86	6110.[illegible]
5	6	23.76	177.72	18	6	268.80	2010.8	32	6	829.58	6205.[illegible]
5	7	24.48	183.15	18	9	276.12	2065.5	32	9	842.39	6301.[illegible]

* Also cubic feet for 1 ft in depth.

Capacity of Cisterns and Tanks

Number of barrels (31½ gal) in cisterns and tanks

Diameter, ft								
5	6	7	8	9	10	11	12	13
23.3	33.6	45.7	59.7	75.5	93.2	112.8	134.3	157.6
[illegible].0	40.3	54.8	71.7	90.6	111.9	135.4	161.1	189.1
32.7	47.0	64.0	83.6	105.7	130.6	158.0	188.0	220.6
37.3	53.7	73.1	95.5	120.9	149.2	180.5	214.8	252.1
42.0	60.4	82.2	107.4	136.0	167.9	203.1	241.7	283.7
46.7	67.1	91.4	119.4	151.1	186.5	225.7	268.6	315.2
51.3	73.9	100.5	131.3	166.2	205.1	248.2	295.4	346.7
56.0	80.6	109.7	143.2	181.3	223.8	270.8	322.3	378.2
60.7	87.3	118.8	155.2	196.4	242.4	293.4	349.1	409.7
65.3	94.0	127.9	167.1	211.5	261.1	315.9	376.0	441.3
70.0	100.7	137.1	179.0	226.6	289.8	338.5	402.8	472.8
74.7	107.4	146.2	191.0	241.7	298.4	361.1	429.7	504.3
79.3	114.1	155.4	202.9	256.8	317.0	383.6	456.6	535.8
84.0	120.9	164.5	214.8	272.0	335.7	406.2	483.4	567.3
88.7	127.6	173.6	226.8	287.0	354.3	428.8	510.3	598.0
93.3	134.3	182.8	238.7	302.1	373.0	451.3	537.1	630.4

Diameter, ft								
[illegible] 8	209.8	238.7	269.5	302.1	336.6	373.0	411.2	451.3
[illegible]9.3	251.8	286.5	323.4	362.6	404.0	447.6	493.5	541.6
[illegible]5.9	293.7	334.2	377.3	423.0	471.3	522.2	375.7	631.9
[illegible]2.4	335.7	382.0	431.2	483.4	538.6	596.8	658.0	722.1
[illegible]9.0	377.7	429.7	485.1	543.8	605.9	671.4	740.2	812.4
[illegible]5.5	419.6	477.4	539.0	604.3	673.3	746.0	822.5	902.7
[illegible].1	461.6	525.2	592.9	667.7	740.6	820.6	904.7	992.9
[illegible]3.6	503.5	572.9	646.8	725.1	807.9	895.2	987.0	1083.2
[illegible].2	545.5	620.7	700.7	785.5	875.2	969.8	1069.2	1173.5
[illegible].8	587.5	668.2	754.6	846.0	942.6	1044.4	1151.5	1263.7
[illegible].3	629.4	716.2	808.5	906.4	1009.9	1119.0	1233.7	1354.0
[illegible].9	671.4	773.9	862.4	966.8	1077.2	1193.6	1315.9	1444.3
[illegible]4	713.4	811.6	916.3	1027.2	1044.6	1268.2	1398.2	1534.5
[illegible].0	755.3	859.4	970.2	1087.7	1211.9	1342.8	1480.4	1624.8
[illegible]5	797.3	907.1	1024.1	1148.1	1279.2	1417.4	1562.7	1715.1
[illegible].1	839.3	954.9	1078.0	1208.5	1346.5	1492.0	1644.9	1805.3

Diameter, ft							
[illegible]3	24	25	26	27	28	29	30
[illegible]3.3	537.1	582.8	630.4	679.8	731.1	784.2	839.3
[illegible]0	644.5	699.4	756.5	815.8	877.3	941.1	1007.1
[illegible].6	752.0	815.9	882.5	951.7	1023.5	1097.9	1175.0
[illegible]3	859.4	932.5	1008.6	1087.7	1169.7	1254.8	1342.8
[illegible]9	966.8	1049.1	1134.7	1223.6	1316.0	1411.6	1510.7
[illegible].6	1074.2	1165.6	1260.8	1359.6	1462.2	1568.2	1678.5
[illegible].2	1181.7	1282.2	1386.8	1495.6	1608.7	1723.0	1846.4
[illegible].9	1289.1	1398.7	1512.9	1631.5	1754.6	1882.2	2014.2
[illegible].6	1396.5	1515.3	1639.0	1767.5	1900.8	2039.0	2182.0
[illegible].2	1503.9	1631.9	1765.1	1903.4	2047.1	2195.9	2343.9
[illegible].9	1611.4	1748.4	1891.1	2039.4	2193.3	2352.7	2517.8
[illegible]5	1718.8	1865.0	2017.2	2175.4	2339.5	2509.6	2685.6
[illegible].2	1826.2	1981.6	2143.3	2311.3	2485.7	2666.4	2853.5
[illegible].9	1933.6	2098.1	2269.4	2447.3	2631.9	2823.3	3021.3
[illegible].5	2041.1	2214.7	2395.4	2583.2	2778.1	2980.1	3189.2
[illegible].2	2148.5	2321.2	2521.5	2719.2	2924.4	3137.0	3357.0

tapering, measure the diameter four-tenths from large end.

Number of U. S. Gallons in Rectangular Tanks

For One Foot in Depth

1 cu ft = 7.4805 gal

Width, ft	Length of tank, ft										
	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
2	29.92	37.40	44.88	52.36	59.84	67.32	74.81	82.29	89.77	97.25	104.73
2.5	....	46.75	56.10	65.45	74.80	84.16	93.51	102.80	112.21	121.56	130.91
3	....	....	67.32	78.54	89.77	100.99	112.21	123.43	134.65	145.87	157.09
3.5	....	....	....	91.64	104.73	117.82	130.91	144.00	157.09	170.18	183.27
4	....	....	....	....	119.69	134.65	149.61	164.57	179.53	194.49	209.45
4.5	....	....	....	....	....	151.48	168.31	185.14	201.97	218.80	235.63
5	....	....	....	....	....	....	187.01	205.71	224.41	243.11	261.82
5.5	....	....	....	....	....	....	....	226.28	246.86	267.43	288.00
6	....	....	....	....	....	....	....	....	269.30	291.74	314.18
6.5	....	....	....	....	....	....	....	....	....	316.05	340.36
7	....	....	....	....	....	....	....	....	....	....	366.54

Width, ft	Length of tank, ft									
	7.5	8	8.5	9	9.5	10	10.5	11	11.5	12
2	112.21	119.69	127.17	134.65	142.13	149.61	157.09	164.57	172.05	179.53
2.5	140.26	149.61	158.96	168.31	177.66	187.01	196.36	205.71	215.06	224.41
3	168.31	179.53	190.75	202.97	213.19	224.41	235.63	246.86	258.07	269.30
3.5	196.36	209.45	222.54	235.63	248.73	261.82	274.90	288.00	301.09	314.18
4	224.41	239.37	254.34	269.30	284.26	299.22	314.18	329.14	344.10	359.06
4.5	252.47	269.30	286.13	302.96	319.79	336.62	353.45	370.28	387.11	403.94
5	280.52	299.22	317.92	336.62	355.32	374.03	392.72	411.43	430.13	448.83
5.5	308.57	329.14	349.71	370.28	390.85	411.43	432.00	452.57	473.14	493.71
6	336.62	359.06	381.50	403.94	426.39	448.83	471.27	493.71	516.15	538.59
6.5	364.67	388.98	413.30	437.60	461.92	486.23	510.54	534.85	550.16	583.47
7	392.72	418.91	445.09	471.27	497.45	523.64	549.81	575.00	602.18	628.36
7.5	420.78	448.83	476.88	504.93	532.98	561.04	589.08	617.14	645.19	673.24
8	....	478.75	508.67	538.59	568.51	598.44	628.36	658.28	688.20	718.12
8.5	....	....	540.46	572.25	604.05	635.84	667.63	690.42	731.21	763.00
9	....	....	....	605.92	639.58	673.25	706.90	740.56	774.23	807.89
9.5	....	....	....	.....	675.11	710.65	746.17	781.71	817.24	852.77
10	....	....	....	.....	.....	748.05	785.45	822.86	860.26	897.66
10.5	....	....	....	.....	.....	.....	824.73	864.00	903.26	942.56
11	....	....	....	.....	.....	.....	.....	905.14	946.27	987.43
11.5	....	....	....	.....	.....	.....	.....	.....	989.29	1032.3
12	....	....	....	.....	.....	.....	.....	.....	.....	1077.2

To find weight of water in pounds at 62° F., multiply the number of gallons by 8⅓.

Example. To find number of gallons in a rectangular tank that is 7.5 ft by 10 ft., the water being 4 ft deep. Look in the extreme left-hand column for 7.5 and opposite to this in the column headed 10 read 561.04, which being multiplied by 4, the depth of water in the tank, gives 2244.2, the number of gallons required.

(a) PLUMBING AND DRAINAGE

Reliable Rules for Plumbing and Drainage. The water-supply of buildings, including the apparatus for heating water, the system of drainage and sewage, and the various fixtures connected therewith, are installed by the plumber, usually in accordance with specifications prepared by the architect and subject to municipal regulations. An efficient and safe system of plumbing is a matter of vital importance. The following may be used as a reliable guide in any locality.

EXTRACTS * FROM THE RULES AND REGULATIONS OF THE DEPARTMENT OF BUILDINGS OF THE CITY OF NEW YORK; ADOPTED APRIL 23, 1912

Definitions of Terms

(12)† The term PRIVATE SEWER is applied to main sewers that are not constructed by and under the supervision of the Department of Sewers.

(13) The term HOUSE-SEWER is applied to that part of the main drain or sewer extending from a point 2 ft outside of the outer wall of building-vault or area to its connection with public sewer, private sewer or cesspool.

(14) The term HOUSE-DRAIN is applied to that part of the main horizontal drain and its branches inside the walls of the building-vault or area and extending to and connecting with the house-sewer.

(15) The term SOIL-PIPE is applied to any vertical line of pipe extending through roof, receiving the discharge of one or more water-closets with or without other fixtures.

(16) The term WASTE-PIPE is applied to any pipe, extending through roof, receiving the discharge from any fixtures except water-closets.

(17) The term VENT-PIPE is applied to any special pipe provided to ventilate the system of piping and to prevent trap-siphonage and back-pressure.

Materials and Workmanship

Soil-Pipe and Vent-Pipe. (19) All cast-iron pipes and fittings must be uncoated, sound, cylindrical, and smooth, free from cracks, sand-holes and other defects, and of uniform thickness and of the grade known in commerce as EXTRA HEAVY.

(20) Pipe, including the hub, shall weigh not less than the following average weights per linear foot:

Diameters	Weights per linear foot, lb
2 in.	5½
3 in.	9½
4 in.	13
5 in.	17
6 in.	20
7 in.	27
8 in.	33½
10 in.	45
12 in.	54

* These numbered paragraphs, from (12) to (174), extracts from Building Regulations, unedited, except in those details which affect typographical uniformity throughout book. Editor-in-chief.

† Paragraph-numbers are the same as those in the Official Regulations. Missing numbers indicate paragraphs purposely omitted.

(22) All joints must be made with picked oakum and molten lead and be made gas-tight. Twelve (12) oz of fine, soft pig lead must be used at each joint for each inch in the diameter of the pipe.

(24) Wrought-iron and steel water-pipes, vent-pipes, waste-pipes and soil-pipes must be galvanized.

(29) All brass pipe for soil-pipes, waste-pipes, and vent-pipes and solder-nipples must be thoroughly annealed, seamless-drawn, brass tubing of standard iron-pipe gauge.

Lead Waste-Pipes. (37) The use of lead pipes is restricted to the short branches of the soil-pipes and waste-pipes, bends, traps, and roof-connections of inside leaders. SHORT BRANCHES of lead pipe shall be construed to mean not more than

8 ft of 1½-in pipe
5 ft of 2-in pipe
2 ft of 3-in pipe
2 ft of 4-in pipe

(38) All connections between lead pipes and between lead and brass or copper pipes must be made by means of WIPED solder joints.

(39) All lead waste, soil, vent, and flush-pipes must be of the best quality, known in commerce as *D*, and of not less than the following weights per linear foot:

Diameters	Weights per linear foot, lb
1¼ in (for flush-pipes only)	2½
1½ in	3
2 in	4
3 in	6
4 and 4½ in	8

(40) All lead traps and bends must be of the same weights and thicknesses as their corresponding pipe-branches. Sheet lead for roof-flashings must be 6-lb lead and must extend not less than 6 in from the pipe, and the joint made water-tight.

(41) Copper tubing when used for inside leader roof-connections must be seamless-drawn tubing not less than 22 gauge, and when used for roof-flashings must be not less than 18 gauge.

Yard, Area and Other Drains

(54) All yards, areas, and courts exceeding 15 sq ft in area must be drained into the sewer. A shaft open at the top and not exceeding 25 sq ft in area, and which cannot be connected in back of a leader, yard, court, or area drain-trap, may be drained into a publicly placed, water-supplied, properly trapped and vented slop-sink.

(59) These drains, when sewer-connected, must have connections not less than 3 in in diameter. They should be controlled by one trap, the leader-trap if possible.

Leaders

(60) Every building shall be kept provided with proper metallic gutters and rain-leaders for conducting water from all roofs in such manner as shall protect the walls and foundations of said buildings from injury. In no case shall the water from any rain-leader be allowed to flow upon the sidewalk or adjoining property, but the same shall be conducted by proper pipes to the sewer. If there be no sewer in the street upon which the buildings front, then the water

leaders shall be conducted by proper pipes below the surface of the
› the street-gutter, or may be conducted by extra-heavy cast-iron
eching cesspool located at least 20 ft from any building. No plumb-
shall discharge into a leeching cesspool.
de leaders must be made of cast iron, wrought iron, or steel, with
tions made gas-tight and water-tight by means of a heavy lead or
rn tubing wiped to a brass ferrule or nipple calked or screwed into

side leaders may be of sheet metal, but they must connect with the
by means of a cast-iron pipe extending vertically 5 ft above the

lers must be trapped with cast-iron running traps so placed as to
zing.
-water leaders must not be used as soil-pipes, waste-pipes or vent-
all any such pipe be used as a leader.

[H]ouse-Sewer, House-Drain, House-Trap and Fresh-Air Inlet

house-drain must properly connect with the house-sewer at a point
of the outer front vault or area-wall of the building. An arched or
opening in the wall must be provided for the drain to prevent
settlement.
house-drain if above the cellar-floor, must be supported at inter-
by 8-in brick piers or suspended from the floor-beams, or be other-
supported by heavy iron-pipe hangers at intervals of not more

eam-exhaust, boiler blow-off, or drip-pipe shall be connected with
in. Such pipes must first discharge into a proper condensing tank,
a proper outlet to the house-sewer outside of the building must be
low-pressure steam-systems the condensing tank may be omitted,
-connection must be otherwise as above required.
ouse-drain and house-sewer must be run as direct as possible,
at least ¼ in per ft, all changes in direction made with proper
ll connections made with Y branches and one-eighth and one-six-

use-Sewer. (74) The house-sewer and house-drain must be at
liameter where water-closets discharge into them. Where rain-
es into them, the house-sewer and house-drain up to the leader-
ust be in accordance with the following table:

Diameter of pipe, in	For a fall of ¼ in per foot, sq ft of drainage-area	For a fall of ½ in per foot, sq ft of drainage-area
3	1 200	1 500
4	2 500	3 200
5	4 500	6 000
6	8 000	10 000
7	12 400	15 600
8	18 000	22 500
9	25 000	31 500
10	41 000	59 000
12	69 000	98 000

(75) Full-size Y and T-branch fittings for hand-hole clean-outs must be provided where required on house-drain and its branches. No clean-out need be larger than 6 in in diameter.

(76) An iron running-trap must be placed on the house-drain near the wall of the house, and on the sewer-side of all connections, except a Y fitting used to receive the discharge from an automatic sewage-lift, oil-separator or a drip-pipe where one is used. If placed outside the house or below the cellar-floor it must be made accessible in a brick manhole, the walls of which must be 8 in thick, with an iron or flagstone cover. When outside the house it must never be less than 3 ft below the surface of the ground.

(79) A FRESH-AIR INLET must be connected with the house-drain just inside of the house-trap and extended to the outer air, terminating with a return-bend, with open end 1 ft above the grade at most available point, to be determined by the superintendent of buildings and shown on plans. The fresh-air inlet-pipe must be of the same diameter as the house-drain. An automatic device approved by the superintendent of buildings may be used when set in a manner satisfactory to him.

Note. The fresh-air inlet and running trap prescribed by Sections 76 and 79 are not required in many cities, and it is better to omit them where not required.

Soil-Pipes, Waste-Pipes and Vent-Pipes

(81) All main, soil, waste or vent-pipes must be of iron, steel, or brass.

(90) The diameters of soil-pipes and waste-pipes must not be less than those given in the following table:

Main soil-pipes	4 in
Main soil-pipes for water-closets on five or more floors	5 in
Branch soil-pipes	4 in
Main waste-pipes	2 in
Main waste-pipes for kitchen-sinks on five or more floors	3 in
Branch waste-pipes for laundry-tubs	1½ in
When set in ranges of three or more	2 in
Branch waste for kitchen-sinks	2 in
Branch waste for urinals	2 in
Branch waste for other fixtures	1½ in

(97) The SIZES OF VENT-PIPES throughout must not be less than the following:

For main vents, 2 in in diameter; for water-closets on three or more floors, 3 in in diameter; for other fixtures on less than seven floors, 2 in in diameter; 3-in vent-pipe will be permitted for less than nine stories; for more than eight and less than sixteen stories, 4 in in diameter; for more than fifteen and less than twenty-two stories, 5 in in diameter; for more than twenty-one stories the size of vent-pipe shall be determined by the superintendent of buildings.

For fixtures other than water-closets and slop-sinks and for more than eight stories, vent-pipes may be 1 in smaller than above stated.

Traps

(101) Every fixture must be separately trapped by a water-sealing trap placed as close to the fixture-outlet as possible and no trap shall be placed more than 2 ft from any fixture.

set of not more than three wash-trays may connect with a single trap, trap of an adjoining sink, provided both sink and tub waste-outlets same side of the waste-line and the sink is nearest the line. When so the waste-pipe from the wash-trays must be branched in below the

ie discharge from any fixture must not pass through more than one reaching the house-drain.

l earthenware traps must have approved heavy brass floor-plates cured to the branch soil-pipe and bolted to the trap-flange and the gas-tight. The use of rubber washers for floor-connections is pro- ll floor-flanges must be set in place and inspected before any water- thereon.

trap shall be placed at the foot of main soil- and waste-pipe lines.

e sizes for traps must not be less than those given in the following

ater-closets	4 in in diam.
op-sinks	2 in in diam.
itchen-sinks	2 in in diam.
ash-trays	2 in in diam.
inals	2 in in diam.
ower-baths	2 in in diam.
her fixtures	1½ in in diam.

leaders, areas, floor and other drains must be at least 3 in in diam-

Water-Closets

enement-houses, lodging-houses, factories, workshops, and all gs the entire water-closet apartment and side walls to a height e floor, except at the door, must be made water-proof with asphalt, netal, or other water-proof material as approved by the superin- ldings.

general water-closet accommodation of any building cannot be cellar nor can any water-closet be placed outside of a building, ce an existing water-closet.

sewer-connected occupied buildings there must be at least one nd there must be additional closets so that there will never be en persons per closet.

ging-houses there must be one water-closet on each floor, and more than fifteen persons on a floor, there must be one additional r every fifteen additional persons or fraction thereof.

-closets and urinals must never be connected directly with or e water-supply pipes, except when flushometer-valves are used, water-closet and urinal-cisterns and automatic water-closets erns are prohibited unless approved by the superintendent of

pper lining of water-closets and urinal-cisterns must not be lighter er.

closet flush-pipes must not be less than 1¼ in and urinal flush- ameter, and if of lead must not weigh less than 2½ lb and 2 lb h-couplings must be of full size of the pipe.

Sinks and Wash-Tubs

(147) In all houses sinks must be entirely open, on iron legs or brackets, without any enclosing woodwork.

(148) Wooden wash-tubs are prohibited, except when used in hotels, restaurants or bottling establishments for washing dishes or bottles. Cement or artificial stone tubs will not be permitted unless approved by the superintendent of buildings.

Testing the Plumbing-System

(171) The entire plumbing and draining-system within the building must be tested by the plumber, in the presence of a plumbing inspector, under a water-test. All pipes must remain uncovered in every part until they have successfully passed the test. The plumber must securely close all openings as directed by the inspector of plumbing. The use of wooden plugs for this purpose is prohibited.

(172) The water-test will be applied by closing the lower end of the main house-drain and filling the pipes to the highest opening above the roof with water. The water-test shall include at one time the house-drain and branches, all vertical and horizontal soil, waste and vent and leader-lines and all branches therefrom to point above the surface of the finished floor and beyond the finished face of walls and partitions. If the drain or any part of the system is to be tested separately, there must be a head of water at least 6 ft above all parts of the work so tested, and special provision must be made for including all joints and connections in at least one test.

(173) After the completion of the plumbing-work, in any new or altered building and before the building is occupied, a final smoke-test must be applied in the presence of the plumbing-inspector. Except that for a building not over six stories in height, a peppermint-test may be applied.

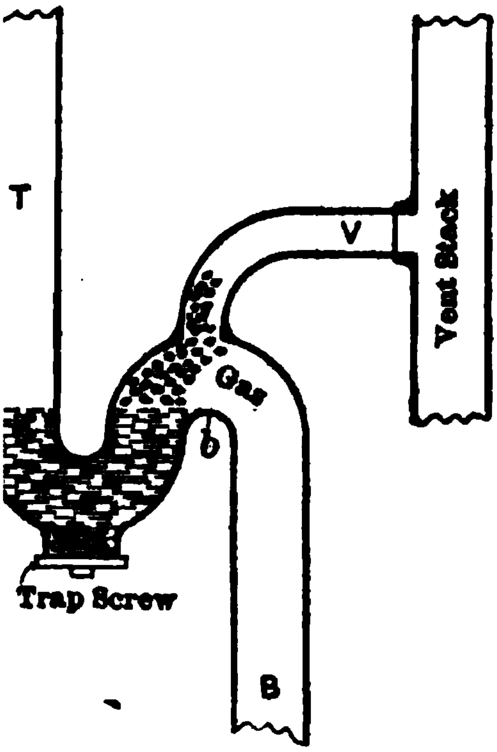
(174) The material and labor for the tests must be furnished by the plumber. Where the peppermint-test is used, 2 oz of oil of peppermint must be provided for each line up to five stories and cellar in height, and an additional ounce of oil of peppermint must be provided for each line when lines are more than five stories in height.

Traps

A trap is a device which permits the free passage of liquids through it, and also of any solid matters that may be carried by the liquid, while at the same time preventing the passage of air or gas in either direction. Traps used for plumbing purposes are shaped so that an amount of water sufficient to close the passage and prevent the passage of air will stand in them at all times. The principle of the common trap is shown in Fig. 7. The pipe *T* receives the waste from a sink or wash-basin, while the lower end *B* connects with the sewer. Sewer-gas rises in pipe *B*, but is prevented from passing to the fixture by the water which stands in the trap. The depth of water through which gas must pass to effect a passage is termed the WATER-SEAL. The water-seal in the trap, Fig. 7, is the distance *S*. All plumbing-pipes which connect with a sewerage-system require to be trapped to prevent sewer-gas from passing through them to the fixture and into the room in which the fixture is located.

Ventilation of Traps. When a considerable body of water rushes down through a pipe it forms a suction, and if the pipe is made air-tight, this suction is often sufficient to prevent enough water remaining in the trap to form a seal, thus leaving an opening for the passage of sewer-gas, as in Fig. 8. By connecting the upper bend of a trap with the outside air by means of a pipe, as at *V*, Fig.

will be stopped, and the water in the pipe *T* will not fall below the outlet at *b*. Several non-siphoning traps have been patented for of obviating the necessity of back-venting, but they are used to a ly limited extent. There are also several varieties of back-pressure



ig. 7. Water-seal of Trap

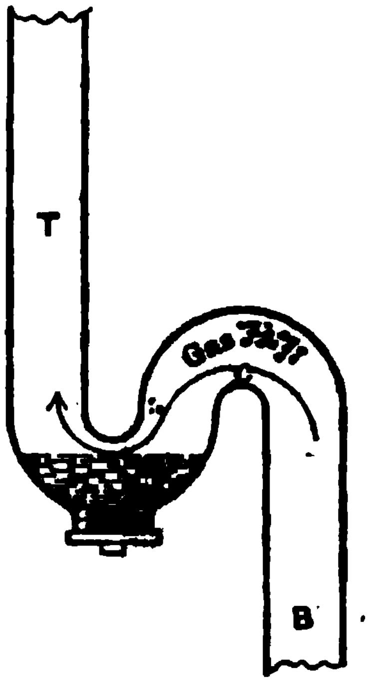


Fig. 8. Water-trap Unsealed

ed to prevent the sewage from flowing back into the house-drain. the nature of check-valves, and are used principally in seaport- tide-water might possibly force the sewage back. The more com- of lead traps used in plumbing, with their trade names, are shown

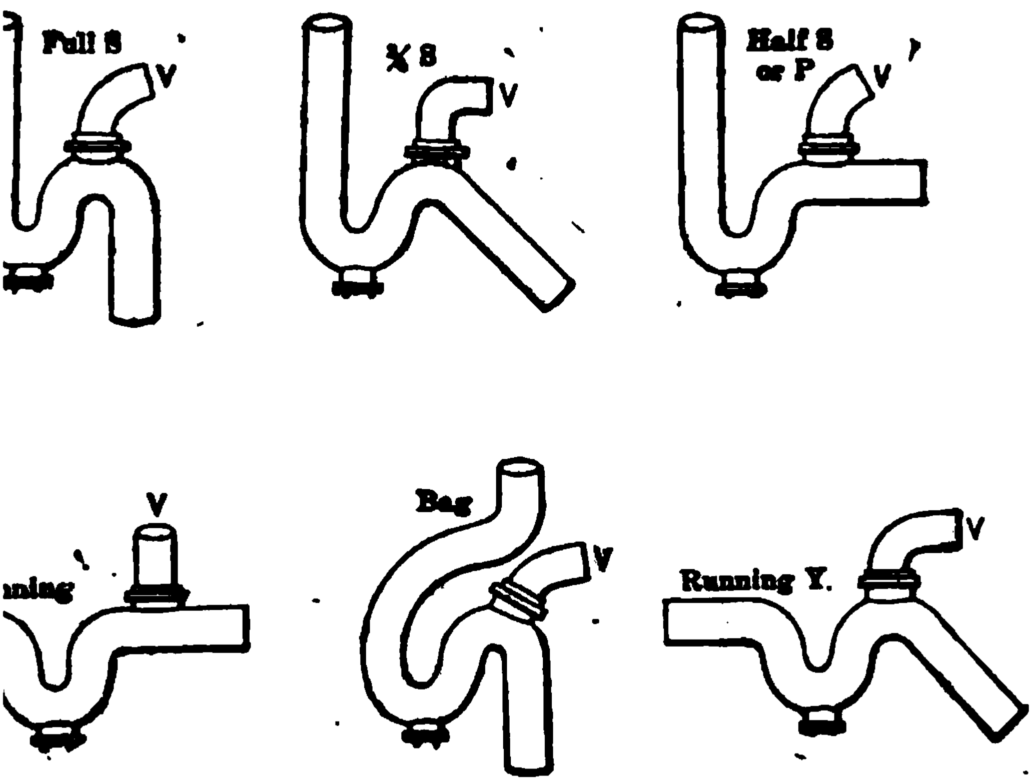


Fig. 9. Types of Traps

he same shapes are also made of cast iron. The pipes marked *V* connections. The drum-trap shown in Fig. 10 has a deeper seal own in Fig. 9, and is commonly used under kitchen-sinks, bath-tubs ys. Drum-traps are not easily siphoned, even when not vented. water-closets are commonly formed in the fixture.

Grease-Traps. The waste-water from kitchen-sinks always contains considerable grease, which if permitted to enter the soil-pipe system is liable to clog the pipes by adhering to the walls. In certain localities grease gives much more trouble than in others, due to the chemical composition of the water. In Colorado and many other places it is necessary to connect the waste from kitchen-

Fig. 10. Drum-trap

Fig. 11. Outdoor Grease-trap

sinks with a large grease-trap, which collects and holds the grease, but permits the water to pass into the sewer system. After a time the accumulated grease fills the trap and must be removed. On account of this it is desirable to use a large trap, and whenever possible it should be placed underground, just outside the house, and as near to the sink as practicable. Grease-traps to be placed underground are commonly made of 24-in vitrified drain-tile or cement pipe, and should be about 4 ft deep. They may also be built of brick in cement mortar. Fig. 11 shows a section through such a grease-trap and the inlet and outlet-pipes. When the sink is in a basement or an upper story, or when the building occupies the entire lot, the grease-trap must be placed under the sink. When so placed, a round lead trap 12 or 14 in in diameter may be used, with a large trap-screw in the top for removing the grease. Fig. 12 shows a section through such a trap and the way in which the connections should be made. A better form of grease-trap is made of cast iron. Some city ordinances require that inside grease-traps shall have a chilling-jacket for the purpose of more perfectly separating the grease and thus preventing any of it from entering the waste-pipes. To be effective, a grease-trap must have a capacity of at least twice the amount of greasy water that will be discharged into it at any one time.

Fig. 12. Lead Grease-trap

es. These may be of lead, brass, galvanized iron, tin-lined lead, Lead pipe offers the least resistance to the flow of water, is easily ny situation, and easy curves are readily made. It is generally re durable underground than galvanized-iron pipe. The grade r STRONG, is the lightest that should ever be used, and when the n from city mains, in which there is a considerable pressure, *AA*, g pipe, should be used. Galvanized-iron pipe is probably more ed than any other material for water-supply pipes in buildings, nickel-plated pipe is required, in which case brass piping is com-Brass pipe used for water-supply should be what is known as IRON-rass piping is preferable to galvanized iron or lead for conveying d is largely used in the better class of buildings. Tin-lined iron s and pipes of block tin are usually considered as offering the ance to corrosion or chemical action, and should always be used ale, beer and other liquors. Tin-lined iron pipe is made by pouring a wrought-iron pipe. While in a fluid state the tin is inseparably ron, and the result is one solid pipe composed of two metals which ORN APART. It is essentially different from iron pipe merely and immeasurably superior to iron pipe lined with a separate will become detached. Its fittings are lined with tin to match. l not injure it, rats will not gnaw it, and thieves will not cut it ot or cold water may stand in block-tin pipes and yet be drawn re and free from poison or rust. Lead-lined pipe is made in the insures delivering the water to the house just as it comes from the ged by the chemical action which often results from contact with pipe.

Drawn Benedict Nickel Tubing is used to some extent for the bing-pipes in high-class residences, office and public buildings. ite metal throughout it cannot rub or wear BRASSY or become is made in all the regular iron-pipe sizes, and necessary fittings the same metal.*

ks. Where the pressure in the street-mains is not great enough fficient volume of water for supplying the fixtures at all times, or rivate water-supply, a tank should be placed in the attic, or ele-6 ft above the highest fixture to be supplied. In some cases the lower story are supplied direct from the street mains, while those ory are supplied from a tank. The advantage of a tank is that it lly from a very small stream, and thus form a reservoir from volume can be drawn in a shorter space of time than could be t from the service-pipes. Storage-tanks should always be pro-overflow-pipe of ample size and when supplied from the street-ly should be controlled by a ball-cock and float. Storage-tanks ze are preferably made of wood lined with planished or tinned lead, zinc or galvanized iron should not be used for lining tanks er for drinking or cooking purposes, and are not as durable as hen the effect on the water need not be considered.

Tank Required will depend largely upon the character of the s supplied from the street-main in which the pressure is fairly not have a capacity exceeding 160 gal. Where the water is e tank by a windmill or hot-air engine, the tank should have a nt for a three or four days' supply at least.

information consult the Benedict & Burnham Manufacturing Company, o.

Amount of Water Required for Various Purposes. The amount of water required for household purposes has been found to be about 25 gal for each person, large or small, but waste will triple that amount sometimes. A horse will drink about 7 gal per day and a cow from 5 to 6 gal per day. A carriage requires from 9 to 16 gal for washing.

Size of Supply-Pipes. The proper diameter of supply-pipes depends upon several considerations, such as the number and size of faucets that are likely to be discharging water at the same time, the urgency of the demand, the length of the pipes and number of angles, and upon the pressure. There is no objection to having a pipe larger than is really necessary, except from the standpoint of cost. Service-pipes should always be one size larger than the tap in the street-main. The following table affords a fair guide for proportioning the supply-branches to plumbing-fixtures. If the pressure is less than 20 lb per sq in the system may be rated as LOW PRESSURE, and if above 20 lb as HIGH PRESSURE.

Supply-branches	Low pressure, in	High pressure, in
To Bath-cocks	¾ to 1	½ to ⅝
Basin-cocks	½	½
Water-closet flush-tank	½	½
Water-closet flush-valve	1¼ to 1½	1¼ to 1½
Sitz or foot-bath	½ to ¾	½
Kitchen sinks	⅝ to ¾	½ to ⅝
Pantry sinks	½	½
Slop-sinks	⅝ to ¾	½ to ⅝
Urinals	⅝ to ¾	½ to ⅝

With high-pressure systems, dwellings of five or six rooms are sometimes, for economy, supplied entirely through ¾-in pipe.

Minimum Diameter of Waste-Pipes. The following are considered as the smallest diameters allowable for waste-pipes. The diameters required in New York City are given on page 1410.

Bath and sink-wastes, 1½ in.

Basin and urinal-wastes, 1¼ in.

Wash-trays, 1½ in from each compartment, entered into 4-in drum-trap and 2-in outlet from trap.

Water-closet trap, 2½ in.

Approximate Spacing for Tacks on Lead Pipes

Size of pipe, in	Vertical pipe, Distance apart, Hot, in	Vertical pipe, Distance apart, Cold, in	Horizontal pipe, Distance apart, Hot, in	Horizontal pipe, Distance apart, Cold, in
½	19	25	14	17
⅝	20	26	15	18
¾	21	27	16	19
1	22	28	17	20
1¼	23	29	18	21
1½	24	30	18	22

on of **Lead Pipe.** The different thicknesses of lead pi
gnated by letters as in Table H, page 1418, but are no
esignated as in Table G, following, which may be consid
epted by dealers.

Table G. Weights and Sizes of Lead Pipe

liber	Weight per foot		Caliber	Wei
	lb	oz		lb
:............	....	6	1½-in Aqueduct...........	3
ine...........	1	4	Extra light.........	3
ict...........	....	8	Light..............	4
ght..........	....	9	Medium...........	5
....	....	12	Strong.............	6
1............	1	....	Extra strong.......	7
....	1	8	Extra extra strong..	9
rong........	2	....	1¾-in Extra light.........	3
ict...........	....	10	Light..............	4
ght..........	....	12	Medium...........	5
....	1	....	Strong.............	6
1............	1	4	Extra strong.......	8
....	1	12	2-in Waste...............	3
....	2	....	Extra light.........	4
rong........	2	8	Light..............	5
tra strong...	3	....	Medium...........	7
ct...........	....	12	Strong.............	8
ht..........	1	4	Extra strong.......	9
....	1	12	Extra extra strong..	10
....	2	....	2½-in Waste...............	4
....	2	8	Light..............	6
ong........	3	....	Medium, 3/16 thick.	8
tra strong...	3	8	Strong, ¼ thick....	11
ict	1	....	Extra strong, 5/16	
ght	1	8	thick.............	14
....	2	....	Extra extra strong,	
1............	2	4	⅜ thick..........	17
....	3	....	3-in Waste.............	4
rong........	3	8	Light..............	6
tra strong..	4	....	Medium, 3/16 thick.	9
ct..........	1	8	Strong, ¼ thick....	12
ht..........	2	....	Extra strong, 5/16	
....	2	8	thick.............	16
t............	1	8	Extra extra strong,	
ht..........	2	....	⅜ thick..........	20
....	2	8	3½-in Waste...............	5
....	3	4	Strong, ¼ thick....	15
....	4	....	Extra strong, 5/16	
rong........	4	12	thick.............	18
tra strong..	5	8	4-in Waste...............	5
ct..........	2	....	Medium...........	10
ht..........	2	8	Strong, ¼ thick....	16
....	3	....	Extra strong, 5/16	
....	3	12	thick.............	22
....	4	12	Extra extra strong,	
ong........	6	....	⅜ thick..........	25
ra strong..	6	12	5-in Waste...............	8

Coils of supply-pipe weigh about 200 lb; aqueduct about 90 lb; suction-pipe, 100 to 180 lb each.

Block-tin pipe is stronger for a given weight per foot than lead pipe or tin-lined lead pipe. As compared with lead pipe its strength is as 3½ to 1.

Tin-lined and lead-lined iron pipe is made with inside diameters of ½, ¾, 1, 1¼, 1½ and 2 in, and in 10-ft lengths, threaded without couplings. Tin-lined and lead-lined fittings are also made (see page 1415).

Weights and Sizes of Sheet Lead

Thickness, in...	1/24	1/20	1/16	1/16 full	5/64	3/32	⅛	⅙	⅕	⅕ full	¼	⅓	⅜
Lb per sq ft.....	2½	3	3½	4	5	6	8	10	12	14	16	20	24

Table H. Thickness and Strength of Lead Pipes

Caliber, in	Mark	Weight per foot, lb oz	Thickness, in	Mean bursting pressure, lb	Safe working pressure, lb	Caliber, in	Mark	Weight per foot, lb oz	Thickness, in	Mean bursting pressure, lb	Safe working pressure, lb
⅜	AAA	1 12	0.18	1 968	492	1	A	4 0	0.21	857	214
⅜	AA	1 5	0.15	1 627	406	1	B	3 4	0.17	745	186
⅜	A	1 2	0.13	1 381	347	1	C	2 8	0.14	562	140
⅜	B	1 0	0.125	1 342	335	1	D	2 4	0.125	518	129
⅜	C	0 14	0.11	1 187	296	1	E	2 0	0.10	475	118
⅜	....	0 10	0.087	1 085	271	1	....	1 8	0.09	325	81
7/16	....	0 9½	0.08	775	193	1¼	AAA	6 12	0.275	962	240
½	AAA	3 0	0.25	1 787	446	1¼	AA	5 12	0.25	823	205
½	....	2 8	0.225	1 655	413	1¼	A	4 11	0.21	685	171
½	AA	2 0	0.18	1 393	343	1¼	B	3 11	0.17	546	136
½	A	1 10	0.16	1 285	321	1¼	C	3 0	0.135	420	105
½	B	1 3	0.125	980	245	1¼	D	2 8	0.125	350	87
½	C	1 0	0.10	782	195	1¼	....	2 0	0.095	322	80
½	D	0 9	0.065	468	117	1½	AAA	8 0	0.29	742	185
½	....	0 10	0.07	556	139	1½	AA	7 0	0.25	700	175
½	....	0 12	0.09	625	156	1½	A	6 4	0.22	608	157
⅝	AAA	3 8	0.23	1 548	387	1½	B	5 0	0.18	506	126
⅝	AA	2 12	0.21	1 380	345	½	C	4 4	0.15	430	107
⅝	A	2 8	0.18	1 152	288	½	D	3 8	0.14	315	78
⅝	B	2 0	0.16	987	246	1½	....	3 0	0.12	245	61
⅝	C	1 7	0.117	795	198	1¾	B	5 0	.....	....	116
⅝	D	1 4	0.10	708	177	1¾	C	4 0	.....	....	93
¾	AAA	4 14	0.29	1 462	365	1¾	D	3 10	0.125	318	79
¾	AA	3 8	0.225	1 225	306	2	AAA	10 11	0.30	611	152
¾	A	3 0	0.19	1 072	268	2	AA	8 14	0.25	511	127
¾	B	2 3	0.15	865	216	2	A	7 0	0.21	405	101
¾	C	1 12	0.125	782	195	2	B	6 0	0.19	360	90
¾	D	1 3	0.09	505	126	2	C	5 0	0.16	260	65
1	AAA	6 0	0.30	1 230	307	2	D	4 0	0.09	200	50
1	AA	4 8	0.23	910	227	...	....	.....	.....	....	....

Weight and Sizes of Pure Block-Tin Pipe

e	Weight per foot, oz	Size inside diameter in	Weight per foot, lb
	4	¾	9, 12, 16
	4, 5, 6	1	12, 16
	4, 5, 6, 8	1¼	20, 28
	4, 5, 6, 8	1½	24 and upwards
	5, 6, 8, 10	2	32 and upwards
	9, 12, 16	....	....

Sewer-Pipe

three kinds of sewer-pipe or drain-pipe offered in the market, (1) VITRIFIED CLAY PIPE, (2) SLIP-GLAZED CLAY PIPE and (3) CEMENT ame of the latter sufficiently indicates what it is without any de-he SLIP-GLAZED CLAY PIPE is made of what is known as FIRE-CLAY, rick clay, which retains its porosity when subjected to the most It is glazed with another kind of clay, known as SLIP, which, when eat, melts, creating a very thin glazing, and which, being A FOREIGN THE BODY OF THE PIPE, is liable to wear or scale off. SALT-GLAZED nade of a clay, which, when subjected to an intense heat, becomes ass-like. It is glazed by the vapors of salt, the salt being thrown ereby creating a vapor which unites chemically with the clay, and g, which will not scale or wear off, and is impervious to the action , steam, or any other known substance. It unites with the clay ner as to form PART OF THE BODY OF THE PIPE, and is therefore Salt-glazed pipe can only be made from clay that will vitrify, subjected to an intense heat will become a hard, compact, non- It should be borne in mind that SLIP-GLAZING is only resorted to are of such a nature that they will not vitrify.

ial of Drain-Pipes should be a hard, vitreous substance; not is would lead to the absorption of the impure contents of the drain, s actual strength to resist pressure, would be more affected by the formation of crystals in connection with certain chemical or would be more susceptible to the chemical action of the con- sewerage.

s Should be Salt-Glazed, as this requires them to be subjected intense heat than is needed for SLIP-GLAZING, and thus secures a l. Cement pipes made without metal reinforcement have not tly strong and durable to be used with confidence in any im- When reinforced with metal, however, they have ample strength, cement sewer-pipes of large diameter are used to a considerable e.

ing the diameter of house-sewers, the table on page 1409 will guide. Storm-sewers should be proportioned to the area drained. n rainfall, as shown by statistics, is about 1 in per hour, except vy storms, equal to 27 225 gal per hour for each acre, or 453 gal acre. Owing to various obstructions, not more than from 50 to fall will reach the drain within the same hour, and allowance for this fact in determining size of storm-sewer required.

Carrying Capacity of Sewer-Pipe

Gallons per minute

Size of pipe, in	Fall per 100 ft							
	1 in	2 in	3 in	6 in	9 in	1 ft	2 ft	3 ft
3	13	19	23	32	40	46	64	79
4	27	38	47	66	81	93	131	163
6	75	105	129	183	224	258	364	450
8	153	216	265	375	460	527	750	923
9	205	290	355	503	617	712	1 006	1 240
10	267	378	463	755	803	926	1 310	1 613
12	422	596	730	1 033	1 273	1 468	2 076	2 554
15	740	1 021	1 282	1 818	2 224	2 464	3 617	4 467
18	1 168	1 651	2 022	2 860	3 508	4 045	5 704	7 047
24	2 396	3 387	4 155	5 874	7 202	8 303	11 744	14 466
27	4 407	6 211	7 674	10 883	13 257	15 344	21 771	26 622
30	5 906	8 352	10 223	14 298	17 714	20 204	28 129	35 513
36	9 707	13 769	16 816	23 763	29 284	33 722	47 523	58 406

Quantities of Cement, Sand and of Cement Mortar for Sewer-Pipe Joints

Prepared by J. N. Hazlehurst

For each 100 ft of sewer (with Portland cement, 375 lb net per bbl)

Size of pipe, in	Length, ft	Mortar, cu yd	Proportions: 1 Cement to					
			1 Sand			2 Sand		
			Cement, bbl	Sand, cu yd	Pipe per bbl cement, lin ft	Cement, bbl	Sand, cu yd	Pipe per bbl cement, lin ft
6	2½	0.003	0.01248	0.00201	803	0.00855	0.00252	1 168
8	2½	0.038	0.15808	0.02546	633	0.10830	0.03192	923
10	2½	0.058	0.24128	0.03886	410	0.16530	0.04872	605
12	2½	0.089	0.37024	0.05963	270	0.25365	0.07476	394
15	2½	0.123	0.51268	0.08241	195	0.35055	0.10332	285
18	2½	0.167	0.69472	0.11189	144	0.47595	0.14018	210
20	2½	0.237	0.98592	0.15879	101	0.67545	0.19908	148
24	2½	0.299	1.24384	0.20033	80	0.85215	0.25116	117
27	3	0.492	2.04672	0.32964	49	1.40220	0.41328	71
30	3	0.548	2.27968	0.36716	44	1.56180	0.46032	64
36	3	0.849	3.53184	0.56883	29	2.41965	0.71316	41

Plumbing Specialties

The Kenney Flushometer. This is a gravity valve designed for flushing all water-closets, urinals and slop-sinks in a building direct from one tank situated in the attic or where most desirable, thus dispensing with the individual overhead tank. The pipe from the main tank is run down to the different floors either exposed or concealed and branches taken off from there to the flushom-

peration of the flushometer is to pull the handle forward, which ain valve off its seat, making a direct connection from the flushom- tank. After the handle is released the valve closes slowly of its against a high or low pressure. It is constructed without springs ers and closes by gravity, is built to stand the hardest service, simple in construction and operation that the same valve is used ements, the only differences in adjustment being those necessary high or low pressure. The flushometer is extensively used for ets in buildings in the Eastern States, including many large office- tories, schools, hospitals, and the better class of residences; also s and yachts.

There are few cities in which the public water-supply is not greatly wholesomeness by being filtered, and in many places filtering is cessary. The filter should be large enough so that the velocity of sing through it will be low and it should be so arranged that the can be reversed and the accumulated impurities washed into a In the country a filter suitable for rain-water may be built un- e filtering process being accomplished by beds of sand and gravel. lings, however, a portable filter located in the basement should be linary sand filter, either pressure or gravity, will clarify water of l impurities, suitable for plunge-baths, and other general uses in To provide a perfectly sterile water, however, the filter must be coagulating apparatus to automatically feed a proportionate dose o the raw water. Those so-called filters which are made to screw e of an ordinary faucet should be considered merely as strainers, hat purpose they soon become foul.

ous Water-Heaters are a great convenience for heating water wash-basins in buildings in which a constant supply of hot water l, and especially in residences where the cooking is done by gas. idrical in shape, made of nickel-plated copper, and are usually plated shelf attached to the wall close to the fixture to be supplied. in in diameter and 30 in high will heat 20 gal of water in eight ost of 1½ to 2 cts with gas at $1 per 1 000 cu ft. A large line of s made by the Instantaneous Water Heating Company, Kala- for both gas and gasoline, although gas is preferable when it he cost of heaters varies from $15 to $45, according to size.

tic Water-Heater which maintains water at any desired tem- t attention, provided the building has a supply of live steam, is B. Clow & Sons, the supply of steam being automatically regu- nostat. It will be found especially desirable in hospitals, hotels, es and public institutions. The heater is made in four sizes, of 1 500, 2 500, 4 000 and 6 500 gal per hour.

Cellar-Drainer * is a simple device for raising water from 6 to tention or power, except a supply of steam or water. It is used draining cellars, wheel-pits, furnace-pits, etc., when they are into the sewer. For such places a box or barrel is sunk so that will run into it, and the drainer is set in this receiver and the dis- to a sink or open drain. The drainer performs its functions by steam under pressure through the drainer-point or jet, thus n which draws the water from the receiver in which it is placed e-pipe, and both the jet-water and cellar-water are discharged

* Manufactured by Jas. B. Clow & Sons.

together. As long as the city water or steam passes through the drainer-pi this suction and discharge continues. The supply of water or steam is tur on or off automatically, so that there is no consumption of city water or ste except when the drainer is removing water. This drainer will operate with pressure of 15 lb or more, the heavier the pressure the greater the amount o dead water discharged. When the drainage-water does not have to be rai more than 10 ft, this is the most economical apparatus that can be used, as th amount of city water consumed is very small. The Climax Drainer is made i six sizes, costing from $25 to $160.

Sewage-Ejectment. Mechanical ejectment of sewage is resorted to in cas where the street-sewer is above the level of the area to be drained. This co dition is found principally in the subbasement-floors of tall buildings, unde ground public-comfort stations and underground passenger-stations. A syste of mechanical ejectment consists of a gravity drainage-system to a receivin tank or sump located in a water-tight pit at the lowest part of the drainag system, and a pump or compressed-air ejector to raise the sewage and dischar it into the street-sewer. There are three types of apparatus used to raise sew age to the street sewers, centrifugal pumps, piston-pumps, and compressed-a ejectors. The compressed-air ejectors, however, are commonly used owing t their numerous advantages. They are automatic and almost noiseless in opera tion, are perfectly odorless, and have but few working parts that can get ou of order. Sewage-ejectment apparatus is generally installed in duplicate s that one set may be cut out of service for cleaning or repairs, without inte rupting the drainage-service.

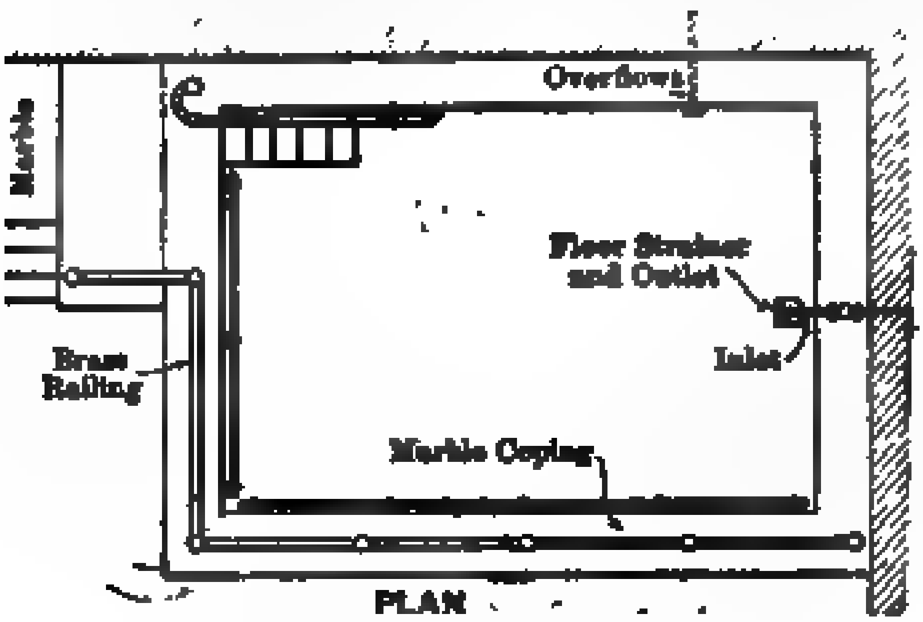
Plunge-Baths

An Example of the Construction and Details of a Small Plung Bath or Swimming-Bath. The following is a description, with illustration of the bath in the house of the Racquet and Tennis Club on Forty-third Stree New York City.*

"The swimming-bath has inside dimensions of 15 by 22 ft and is about 9 in total depth. It was built in a pit about 19 by 26 ft and about 8 ft deep belo the main excavation, which was blasted out of solid rock. A concrete inve 1 ft or more in thickness was laid over the bottom, serving as a footing o which the 12-in walls of common red brick were laid in cement. They we built close to the rough vertical faces of the excavation, and the spaces behin them were filled with concrete or cement mortar or were flushed with grou Then on the inner surface of the walls and on top of the concrete bottom linin a waterproofing of six layers of felt with lapped joints was mopped on with ho tar and flashed around the iron outlet-pipe, which also had a wide calked lea flange extending between the layers of felt. On the bottom of this water-pro coat an 8-in inverted segmental flat floor-arch of common brick was laid, and o its skewbacks 4-in vertical brick walls were built against the water-proofed side The bottom was then lined with vitrified white tile and the sides were faced wi English white enameled brick. The tops of the walls were coped with bevel and molded white-marble slabs which are about 2 ft above the floor-level an are surmounted at one side and one end by a low heavy rail with twisted orn mental posts, all of brass. A similar horizontal hand-rail is carried along th inside wall of the bath just above water-level and a curved brass hand-rail fastened to the wall above the narrow brick-and-marble stairs at one end. T

* The illustrations and accompanying descriptions are taken by permission from t Engineering Record of Nov. 3, 1900.

occupies one corner of the room and its elevated marble plat-
ntirely across it, forming a diving-platform which is reached by
ps. All the water-supply is filtered and it can be warmed by in-
nto the delivery-pipe at the filter. The water enters through the
end of a 2-in brass pipe projecting a foot or more through the
top of the bath and delivering a solid jet unless it is reduced by
valve or is formed into a fan-shaped cascade by means of a



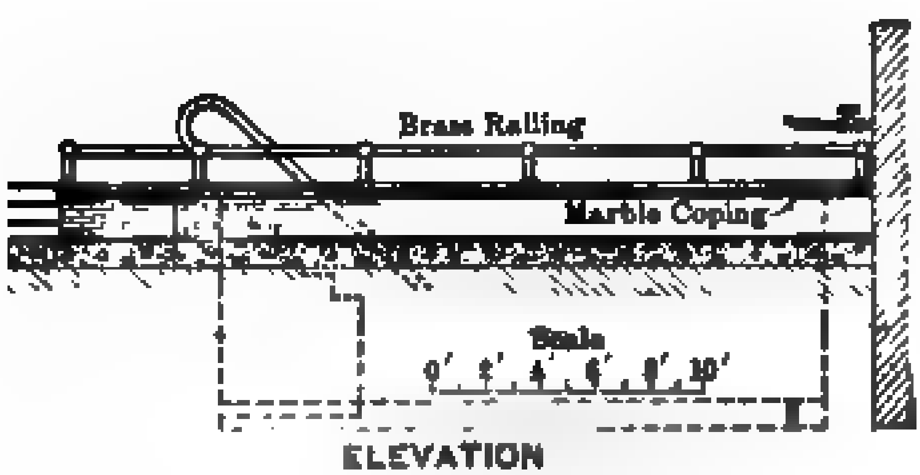


Fig. 13. Plunge-bath

hich can be screwed in the open end of the pipe When the
ed a small stream of water is constantly admitted and causes
e circulation and corresponding overflow, and the entire con-
d out and the bath cleaned every two or three days. There
s, an open one about 8 ft above the bottom and a valved one
L. W. Eidlitz was the architect of the house and the water-
e by the T. New Construction Company."

Plumbing. Figs. 14, 15 and 16 show the symbols suggested
ans and Specifications" for designating plumbing-work on plans

and details, and generally accepted for the purpose. It is just as nece to have conventional symbols to indicate plumbing-work and fixtures, as to have symbols to show-windows, doors, steps, partitions and other str

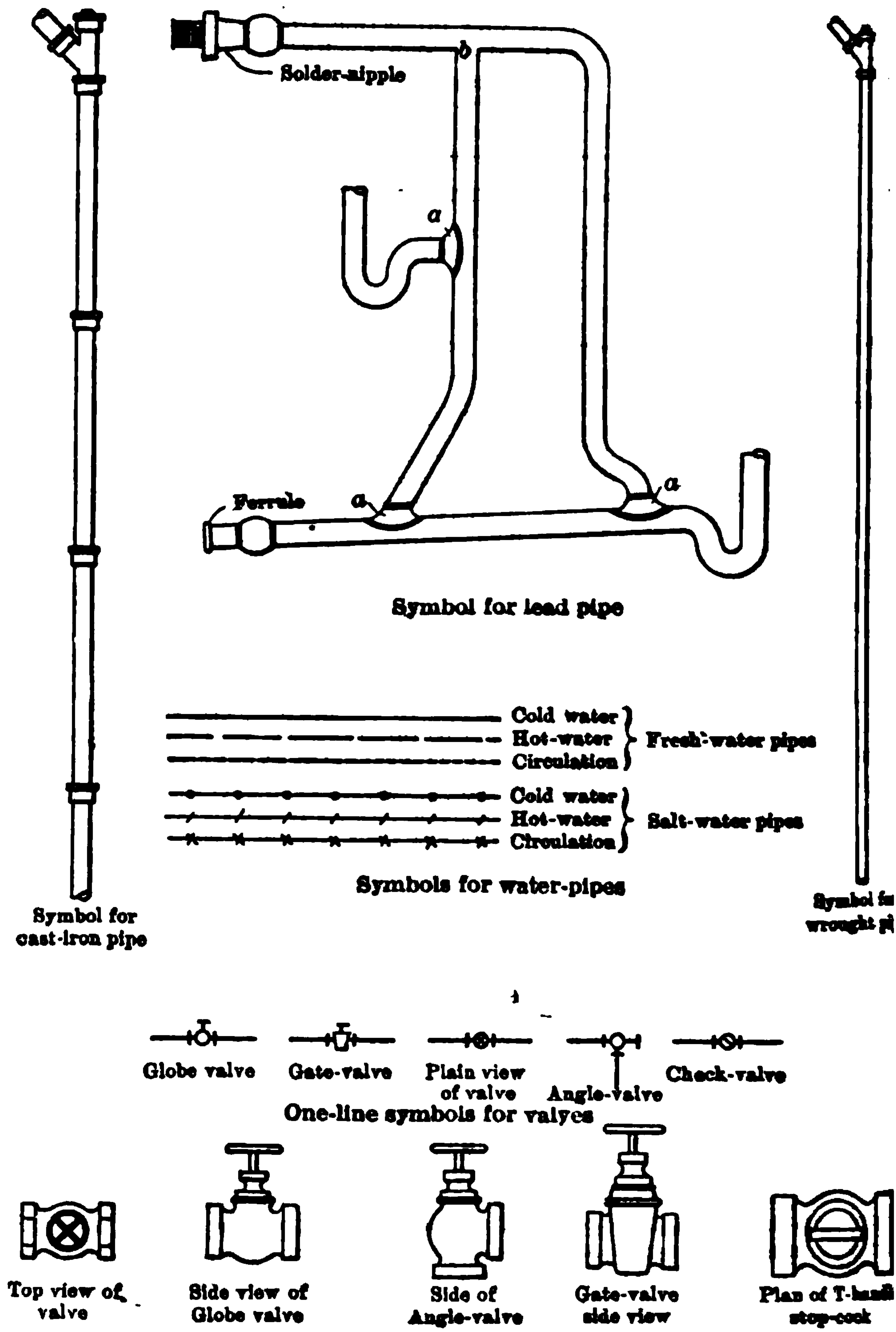


Fig. 14. Symbols for Plumbing-pipes and Valves

tural details on architectural drawings. Before these symbols became gener used there was no uniformity in the drawing of plumbing-plans, and this of standards often led to serious confusion. For instance, if plans from

were examined, the chances were that on no two of them would ave been alike. Further, plans prepared in the same office at

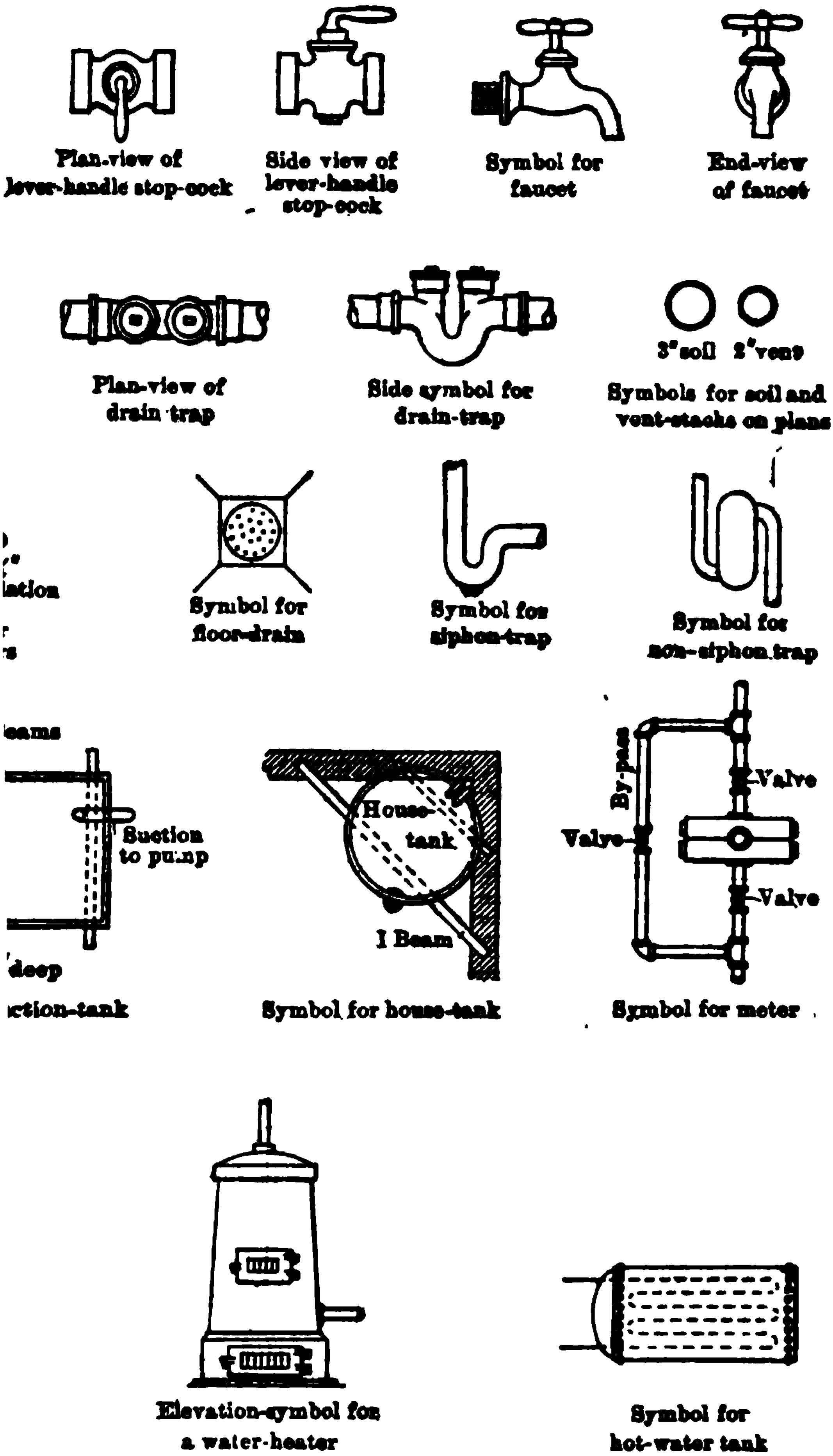


Fig. 15. Miscellaneous Plumbing-symbols

me set of plans on which several different draughtsmen had show as many different symbols for a water-closet or lava- workmen engaged on the drawings. That was rather con-

fusing to plumbers who had to take off quantities from the plans; for, ofte
the symbols were so strange and bore so little resemblance to the fixtures a
apparatus that some of them were liable to be overlooked. It is owing to thi

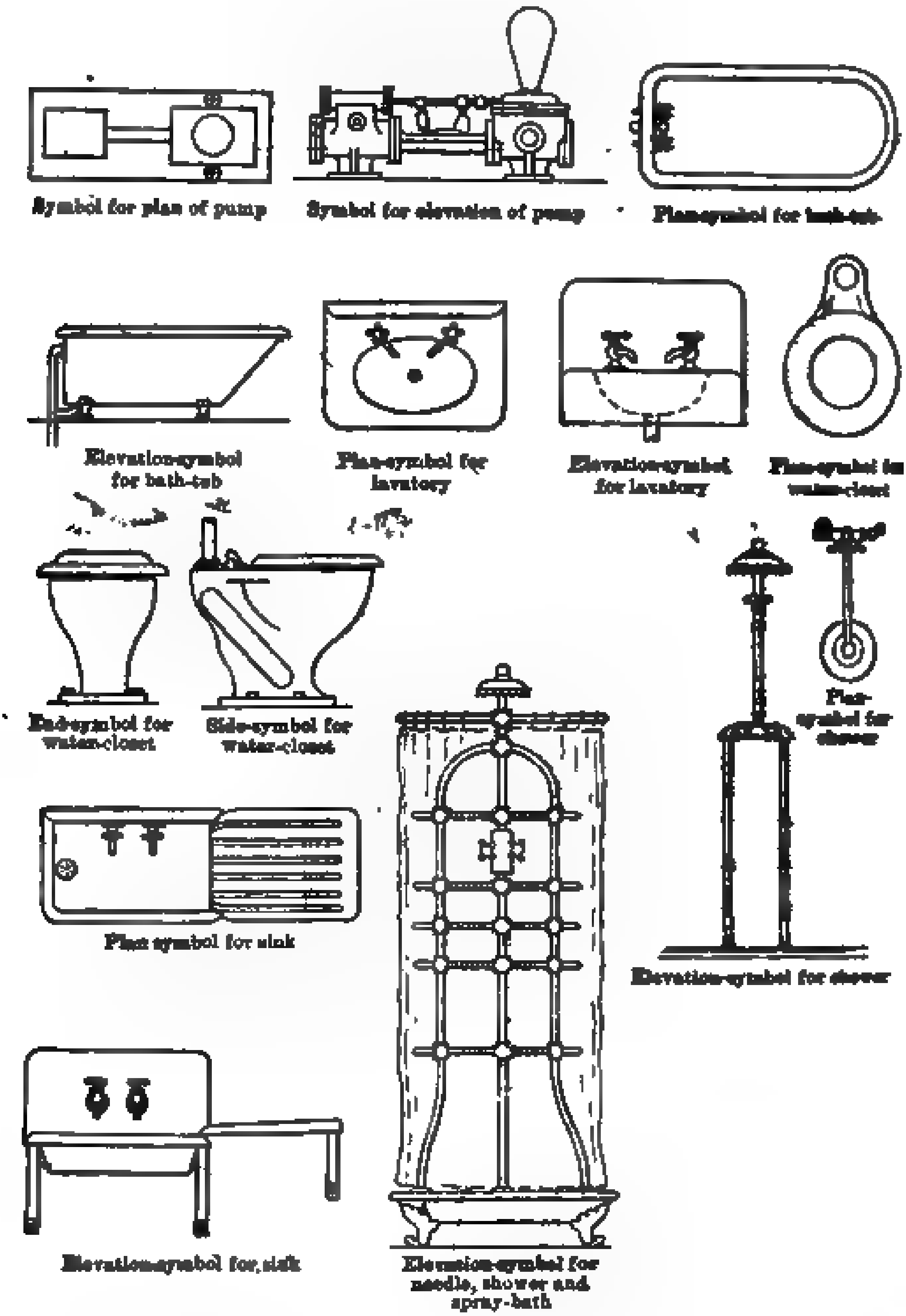


Fig. 16. Symbols for Plumbing-fix

uncertainty wherever it exists from this cause, that
prices in the bids submitted, and all of them are
amount of work to be done. To avoid confusion an
standard symbols should be used.

n of Soil and Waste-Stacks. In tall buildings, provision should the soil-stacks and connections to take care of the expansion, con-ttlements, swaying and other movements of the building. This s no inconsiderable amount, in some localities the settlement alone o as much as 5 in when the foundations are not carried to bed-rock. for instance, most of the sky-scrapers which were built on com-indation-beds are out of plumb and lean far out over the plumb-building in particular leaned so that the top was 30 in outside of he foundation." Most of the earlier heavy buildings there erected foundations" are carried on jacks, and periodically jacked up as ccurs. When the building finally comes to rest, the jacks are d the walls filled in with masonry. The settlement which takes ange in such buildings from 3 to 5 in. These various move-nsion, contraction, settlement, racking out of plumb, also sway-buildings as they follow the sun in its course from East to rove destructive to steam-pipes and plumbing-pipes if provision to take care of them. Steam-pipes always have expansion-loops, y recently that the proper attention has been given to soil and and pipes; and fter as many as osets in one build-en through faulty or rigid connec-remedy is to put nts (Fig. 17) in f vent-stacks of s, and to connect all water-closets to the soil-pipes by means collapsible connections which will stretch, collapse, or stretch on collapse on the other, according to the stress to which they are These flexible fittings should be placed as close to the closets as should be used also in connection with slop-sinks and inside rain-inside rain-leaders the number of corrugations can be increased to the height of the building. Ordinary stock fittings have a ut 2 in. That is, they will stretch about 1 in and collapse 1 in. ers in tall buildings, however, greater range than that is desirable. ions would be sufficient for a rain-leader in an ordinary building ft in height; then, for taller buildings, it is well to allow an extra or each additional 100 ft or fraction thereof. The flexibility of can be seen in the accompanying illustrations of Fig. 17.

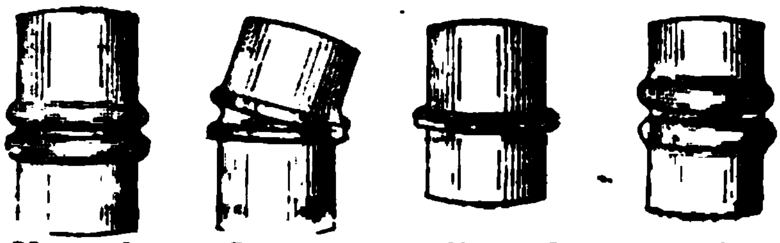


Fig. 17. Expansion-joints

in Buildings. Ninety-seven per cent of buildings erected have -construction, and the floor-joists, when they dry out, shrink. use of many thousands of closets being broken annually, and the the seal at the closet-connection of those which are not broken, re provided with a flexible floor-flange or fitting. The amount of floor-beams of different depths, can be found in the following d from information furnished by the United States Government, f Agriculture, Division of Forestry, in Bulletin No. 10. Besides of the individual tiers of joists, there is the multiple shrink-e tiers when bearing-partitions, supporting the joists at the building, rest on sills at each floor which are laid on top of tead of extending down through to the plate which supports the When the framing is properly done, there is only the shrinkage of beams to take into consideration. When improperly framed,

there might be three or four shrinkages affecting the top floor of the buildin Even though the timbers are dry and seasoned when put in, by the time th plasterers are through the joists are wet and swollen from the moisture in th plaster and from the rain which saturates the timbers before the building is e closed. It is safe to assume, therefore, that a 12-in joist will shrink almost in, and an 18-in joist about ¾ in.

Table of Shrinkage of Timbers

Depth of green or wet timber, in	Amount lost by shrinkage, 4%, in	Depth of timber when dry, in
6	0.24	5.76
8	0.32	7.68
10	0.40	9.60
12	0.48	11.52
14	0.56	13.44
16	0.64	15.36
18	0.72	17.28
20	0.80	19.20

Floor-Connections for Water-Closets. No water-closet can be considere sanitary which depends upon a PUTTY-JOINT, slip-joint, rigid-gasket joint c rigid connection of any kind for a seal. Improved metal-to-metal floor-flang now cost no more than rigid-gasket joints formerly did, and they are flexibl water-tight, will remain permanently tight, and protect the closets from bei broken by shrinkage or other movement of the building or piping. The onl way to get a perfectly sanitary water-closet is to specify a flexible, metal-t metal, closet floor-flange with it.

Expansion of Hot-Water Pipes. In all tall buildings expansion-loo ought to be placed in both the hot-water and the circulation-pipes, to perm the expansion and contraction of the lines without injury to the system. The loops are usually from 6 to 8 ft long, made up with elbows, and extend into th floor of the building. Generally the hot-water and circulation-pipes are su ported midway between loops so that they can expand both up and down. Th length that water-pipes will expand depends upon the degree to which they a heated, and the materials of which the pipes are made. The first of the follo ing three tables gives the expansion of cast-iron pipes, the second the expansio of wrought-iron pipes, and the third the expansion of brass pipes.

Expansion of Cast-Iron Pipes

Temperature of air when pipe is fitted, degrees F.	Length of pipe when fitted, ft	Length of pipe when heated to							
		215° F.		265° F.		297° F.		338° F.	
		ft	in	ft	in	ft	in	ft	in
0	100	100	1.59	100	1.96	100	2.20	100	2.5[illegible]
32	100	100	1.36	100	1.65	100	1.96	100	2.2[illegible]
64	100	100	1.12	100	1.43	100	1.73	100	2.0[illegible]

Expansion of Wrought-Iron Pipe

Length of pipe when fitted, ft	Length of pipe when heated to							
	215° F.		265° F.		297° F.		338° F.	
	ft	in	ft	in	ft	in	ft	in
100	100	1.72	100	2.21	100	2.31	100	2.70
100	100	1.47	100	1.78	100	2.12	100	2.45
100	100	1.21	100	1.61	100	1.87	100	2.19

Expansion of Brass Pipe

Length of pipe when fitted, ft	Length of pipe when heated to							
	215° F.		265° F.		297° F.		338° F.	
	ft	in	ft	in	ft	in	ft	in
100	100	2.58	100	3.18	100	3.56	100	4.05
100	100	2.19	100	2.79	100	3.18	100	3.67
100	100	1.81	100	2.41	100	2.79	100	3.28

[H]ard Water for Domestic Use. In many parts of the country [T]EMPORARILY HARD, PERMANENTLY HARD or both TEMPORARILY [...]TLY HARD. This is due to the fact that in those regions the [...]k is limestone, and in percolating through the limestone the [...]riginally was soft, dissolves carbonates and sulphates of lime or [...] the rock. The solvent capacity of water for lime and magnesia [...] the water is cold than when it is hot. Therefore, deep-well [...]ne-regions is usually saturated with lime or magnesia, and when [...]-tanks or boilers the point of saturation is lowered and lime is liberated in the form of hard scale or incrustation. The effect [...]ation is to shorten the life of the boiler and decrease the efficiency [...]ile in use. It is estimated that:

1/16-in lime-scale means a loss of 13% of fuel.
1/8-in lime-scale means a loss of 22% of fuel.
1/4-in lime-scale means a loss of 38% of fuel.
3/8-in lime-scale means a loss of 50% of fuel.
1/2-in lime-scale means a loss of 60% of fuel.
3/4-in lime-scale means a loss of 91% of fuel.

[...]e probably a little high, but making due allowance, the table [...]w the loss due to the use of hard water. In the laundry the [...]ption of soap to soften hard water is a further item of expense. [...]t 1 lb of soap to soften 100 gal of moderately-hard water, [...] required for washing after the water has been softened. [...]nse, hard water forms an insoluble curd when washing which [...]larly annoying to hotel-guests; therefore, it is advisable to [...]ter for large hotel-buildings, laundries and for many indus- Permanently hard waters contain sulphates of lime or mag- [...]rily hard waters contain carbonates of lime or magnesia.

Temporarily and permanently hard waters contain both carbonates an sulphates of lime or magnesia. Temporarily hard waters are softened b adding lime-water to the raw water to remove the carbonates of lime. Th is known as the CLARK PROCESS. Permanently hard waters are softe by the PORTER PROCESS, which consists of adding soda-ash to the raw wate Stock types of apparatus are manufactured for this purpose, and may be h with capacities of any required amount.

Heating Water with Steam-Coils. The following constants will be foun convenient for proportioning steam-coils for heating water:

W = gallons of water to be heated.
$W \div 10$ = sq ft of iron pipe-coil required for exhaust-steam.
$W \div 15$ = sq ft of copper pipe-coil required for exhaust-steam.
$W \times 0.07$ = sq ft of iron pipe-coil for 5 lb pressure-steam.
$W \times 0.045$ = sq ft of copper pipe-coil for 5 lb pressure-steam.
$W \times 0.05$ = sq ft of iron pipe-coil for 25 lb steam-pressure.
$W \times 0.035$ = sq ft of copper pipe-coil for 25 lb steam-pressure.
$W \times 0.04$ = sq ft of iron pipe-coil for 50 lb steam-pressure.
$W \times 0.25$ = sq ft of copper pipe-coil required for 50 lb steam-pressure.
$W \times 0.03$ = sq ft of iron pipe-coil required for 75 lb steam-pressure.
$W \times 0.02$ = sq ft of copper pipe-coil required for 75 lb steam-pressure.

Capacity of Water-Backs. The average size of water-back having abo 110 sq in, or about ¾ sq ft of exposed surface, will heat to the ordinary tempe ature of domestic hot water, 180° F., about 21 gal of water an hour. It w heat about 17 gal of water to the boiling-point with an ordinary fire. With fire such as is used for roasting, washing, or baking, a water-back of this same si will heat about 23 gal of water to the boiling-point, or 27 gal to a temperatu of 180° F. Wrought-iron pipe heating-coils will heat from 30 to 40 gal of wat under the same conditions, and copper pipes will heat from 45 to 60 gal per ho for each square foot of surface exposed to the fire. In calculating the heati capacity of water-backs or coils, the average temperature of the water is take Thus, if water at 60° is heated to 200° F., the average temperature of the wat would be $(60 + 200) \div 2 = 130°$ F., and the range of temperature through whi it is heated would be $200 - 60 = 140°$ F.

Value of Pipe-Covering. Hot-water pipes and hot-water tanks wh uncovered lose by radiation from their surface about 13 heat-units per min per square foot of surface. To prevent this loss of heat and consequent ext consumption of coal, hot-water pipes, circulation-pipes and hot-water tanks large institutions are generally covered with some non-heat-conducting materi The value of pipe-covering is not proportional to its thickness. Sectional pip coverings average about 1⅛ in in thickness and reduce the loss by radiati about 90%. Doubling the thickness of pipe-covering saves only about anoth 5% of heat-loss. In specifying covering for pipes and boilers, therefore, a thic ness of 1⅛ in will be sufficient. Carbonate of magnesia is a very poor conduc of heat. Therefore, it is a good material for covering hot-water pipes. Carb ate of lime, on the other hand, is not a good covering material, althoug often masquerades as carbonate of magnesia. When magnesia pipe-coveri specified, therefore, it is well to require a composition containing from 80 to 9 of magnesia, and require a test to be made at the expense of the contractor, by a chemist named by the architect. The following coverings are the materials for hot-water pipes, in the order in which they are named. Nonp Cork, Magnesia, Asbestos Air-Cell and Imperial Asbestos.

(3) ILLUMINATING-GAS AND GAS-PIPING*

of Gas. Five varieties of gas are now commonly used for light-
ting, namely:

Gas, which is made by heating bituminous coal in air-tight retorts.
most common variety of gas furnished for the lighting of cities and

-Gas, which is made usually from anthracite coal and steam, and
nsively used in Eastern cities. Gas made by this process contains
han good coal-gas, and consequently does not give as bright a light,
urns perfectly in heating-burners. When used for lighting purposes
l in carbon by vaporizing a quantity of petroleum by heat and in-
to the hot gas before it leaves the generator. Pure water-gas is
as less odor than coal-gas.

l Gas is obtained from holes or wells which are drilled in the ground.
where it can be obtained it furnishes cheap light and fuel. The
btained in the hard-coal regions develops more heat per cubic foot in
any other kind of gas except acetylene. Natural gas is usually under
ure in the street-mains and house-pipes than manufactured gas.

ne-Gas. Used almost exclusively for the lighting of isolated
for public buildings in towns or cities where there is no public gas-
commonly generated on the premises. It is formed by bringing
alcium carbide in contact. Calcium carbide is produced by the
ion of coke and lime. It is now a commercial article produced in
ies and sold at a moderate price. It is a very hard substance like
has a very slight odor, will not burn or explode, and can be handled
ity with perfect safety. The fact that carbide begins to disintegrate
acetylene at the slightest touch of moisture makes it practicable to
gas in small quantities for single buildings.

Generating Acetylene-Gas. The satisfactory production of acety-
ires a generator which shall feed carbide of sufficient size and weight
l a sufficient depth under the water in the generator-chamber to
s and proper washing. The carbide-chamber must be so arranged
l that no gas can return to it to be wasted when the chamber is
ermeate the house with its smell. It must feed carbide loosely
small quantities, in order to provide for perfect coolness by free
er to all of the carbide. It must work automatically and with
ainty. Acetylene-gas to be pure must be thoroughly washed.
lene, as with any other illuminating-gas, means a discoloration of
ninished illuminating power, clogging of pipes and burners with
ther foreign matter, and smoky burners, causing blackening of
arnished and soiled woodwork and upholstery. It is now gener-
at the requirements above outlined can be attained only by a
he plunger-type. Portable generators which may be set in the
nent of any building are manufactured in great variety; it is esti-
o 000 acetylene-gas generators are now in use in the United States.
le in sizes of 5, 10, 15, 20 and up to 500-lights capacity. In all
oping carbide into water there should be a connection open from
olding receptacle to the safety-vent run out of doors from the
t is claimed that for a given degree of illumination, acetylene is
dollar gas. A large residence may be lighted for about $2.50 a

* See, also, Lighting and Illumination of Buildings, page 1437.

month. To develop the full illuminating power of the gas it is necessary to use a burner-tip having the thinnest slit obtainable, the illuminating power of the gas being about fifteen times that of coal-gas, for the same consumption. The light is a clear white, very nearly resembling sunlight in color and diffusiveness, with none of the red of the incandescent lamp, the orange of the ordinary gas-flame, or the green tone of the incandescent mantle; and it possesses the quality, unique among artificial illuminants, of reproducing even the most delicate shades of color as faithfully as sunlight. Even when used with mantle-burners, as it may be with great economy, acetylene-light presents a strong dissimilarity from ordinary gas under the same conditions. Acetylene corrodes silver and copper, but does not affect brass, iron, lead, tin, or zinc. A government specification for a complete apparatus for acetylene-gas was published in Engineering News of Feb. 4, 1904.

(5) **Gasoline-Gas** is a mixture of gasoline vapor with air. It is never piped but is generated close to the burner, and is seldom used for lighting except for street stands, and the like. It is much used for fuel, however.

Gasoline changes from the liquid to the gaseous form under ordinary atmospheric pressure, at temperatures above 40° F., the evaporation being very slow at 40°, quite rapid at 70°, and furious at 212°. If a tank containing liquid gasoline is left open to the air, the liquid will all pass away in the form of gas.

Although generally considered dangerous, it is only so when carelessly or ignorantly handled. To produce 1 000 cu ft of gas of good quality requires about 4½ gal of the best grade of gasoline. An ordinary burner consumes about 5 cu ft per hour.

Piping a House for Gas * †

General Principles and Requirements. Ordinary wrought-iron pipe, such as is used for steam or water, is suitable and proper for all kinds of gas. Galvanized malleable-iron fittings, in distinction from plain iron, are very superior. The coating of zinc inside and out effectually and permanently covers all blow-holes, makes the work solid and durable, and avoids the use of perishable cement. Before the pipe is placed in position it should be looked and blown through. It is not infrequently obstructed, and this precaution will save much damage and annoyance. What is known as gas-fitters' cement never should be used. It cracks off easily, in warm places it will melt and it can be dissolved by several different kinds of gas. Nothing but solid metals is admissible for confining gas of any kind. When pipes under floors run across floor-timbers, the latter should be cut into near their ends, or when supported on partitions, and not near the middle of spans. It is evident that a 10-in timber notched 2 in in the middle is no stronger than an 8-in timber. All branch outlet-pipes should be taken from the sides or tops of running lines. Bracket-pipes should run up from below, and not drop from above. Never drop a center pipe from the bottom of a running line. Always take such outlet from the side of the pipe. The whole system of piping must be free from low places or traps, and decline toward the main rising pipe, which should run up in a partition as near the center of the building as is practicable. It is obvious that where gas is distributed from the center of a building, smaller running lines of pipe will be needed than when the main pipe runs up on one end. Hence, timbers will not

* Circular issued by the Gilbert & Barker Manufacturing Company.

† See, also, Lighting and Illumination of Buildings pages 1437 to 1456.

deep cutting, and the flow of gas will be more regular and even. For eason in large buildings, more than one riser may be advisable. When ; has different heights of post, it is always better to have an in- rising pipe for each height of post, than to drop a system of piping igher to a lower post, or to grade to a low point and establish . Drip-pipes in a building should always be avoided. The whole piping should be so arranged that any condensed gas will flow back he system and into the service-pipe in the ground. All outlet-pipes so securely and rigidly fastened in position that there will be no possi- heir moving when the gas-fixtures are attached. Center pipes should solid support fastened to the floor-timbers near their tops. The pipe : securely fastened to the support to prevent lateral movement. The : must be perfectly plumb, and pass through a guide fastened near the of the timbers, which will keep them in position despite the assault s, masons and others. In the absence of express directions to the , outlets for brackets should generally be 5 ft 6 in high from the floor, at it is usual to put them 6 ft high in halls and bath-rooms. The upright ould be plumb, so that the nipples that project through the walls will be The nipples should project not more than ¾ in from the face of the g. Laths and plaster together are usually ¾ in thick; hence the should project 1½ in from the face of the studding. Drop center pipes roject 1½ in below the furring, or timbers if there is no furring, where own that there will be no stucco or centerpieces used. Where center- re to be used, or where there is a doubt whether they will be or not, e drop-pipes should be left about a foot below the furring. All pipes roperly fastened, the drop-pipe can be safely taken out and cut to the ngth when gas-fixtures are put on. Gas-pipes should never be placed on toms of floor-timbers that are to be lathed and plastered, because they ccessible in the contingency of leakage, or when alterations are desired, -fixtures are insecure. The whole system of piping should be proved to nd gas-tight under a pressure of air that will raise a column of mercury h in a glass tube. The pipes are either tight or they leak. There is no ground. If they are tight the mercury will not fall a particle. A piece r should be pasted on the glass tube, even with the mercury, to mark its while the pressure is on. The system of piping should remain under r at least a half-hour. It should be the duty of the person in charge of nstruction of the building to thoroughly inspect the system of gas-fitting; as much so as to inspect any other part of the building. He should know personal observation that the specifications are complied with. After satisfied that the mercury does not fall he should cause caps on the out- be loosened in different parts of the building, first loosening one to let ir escape, at the same time observing if the mercury falls, then tightening repeating the operation at other points. This plan will prove whether the are free from obstruction or not. When he is satisfied that the whole s properly and perfectly executed, he should give the gas-fitter a certificate t effect.

: following requirements from specifications published by the Denver Gas lectric Company are worthy of attention. Always use fittings in making ; do not bend pipe. Do not use unions in concealed work; use long screws ht-and-left couplings. Long runs of approximately horizontal pipe must nly supported at short intervals to prevent sagging.

Rules and Table for Proportioning Sizes of House-Pipes for Gas*†

Rules Governing Sizes of Gas-Pipes. The table on page 1435 is based on the well-known formula for the flow of gas through pipes. The friction, and therefore the pressure necessary to overcome the friction, increases with the quantity of gas that goes through, and as the aim of the table is to have the loss in pressure not exceed 1/10 in water-pressure in 30 ft, the size of the pipe increases in going from an extremity toward the meter, as each section has an increasing number of outlets to supply. The quantity of gas the piping may be called on to pass through is stated in terms of 3/8-in outlets, instead of cubic feet, outlets being used as a unit instead of burners, because at the time of first inspection the number of burners may not be definitely determined. In making the table, each 3/8-in outlet was assumed to require a supply of 10 cu ft per hour. In using the table observe the following rules:‡

(1) No house-riser shall be less than 3/4 in. The house-riser is considered to extend from the cellar to the ceiling of the first story. Above the ceiling the pipe must be extended of the same size as the riser, until the first branch line is taken off.

(2) No house-pipe shall be less than 3/8 in. An extension to existing piping may be made of 1/4-in pipe to supply not more than one outlet, provided said pipe is not over 6 ft long.

(3) No gas-range shall be connected with a smaller pipe than 3/4 in.

(4) In figuring out the size of pipe, always start at the extremities of the system, and work TOWARD the meter.

(5) In using the table, the lengths of pipe to be used in each case are the lengths measured from one branch or point of juncture to another, disregarding elbows or turns. Such lengths will be hereafter spoken of as SECTIONS. No change in size of pipe may be made except at branches or outlets, each section therefore being made of but one size of pipe.

(6) If any outlet is larger than 3/8 in it must be counted as more than one, in accordance with the schedule below:

Size of outlet, inches...........	1/2	3/4	1	1 1/4	1 1/2	2	2 1/2	3	[illegible]
Value in table..................	2	4	7	11	16	28	44	64	[illegible]

(7) If the exact number of outlets given cannot be found in the table, take the next larger number.

(8) If, for the number of outlets given, the exact length of the section which feeds these outlets cannot be found in the table, the next larger length, corresponding to the outlets given, must be taken to determine the size of pipe required. Thus, if there are eight outlets to be fed through 55 ft of pipe, the length next larger than 55 in the eight-outlet line in the table is 100, and as this is in the 1 1/4-in column, that size pipe would be required.

(9) For any given number of outlets, do not use a smaller size pipe than the smallest size that contains a figure in the table for that number of outlets. Thus, to feed 15 outlets, no smaller size pipe than 1 in may be used, no matter how short the section may be.

(10) In any piping-plan, in any continuous run from an extremity to the meter, there may not be used a longer length of any size pipe than found in the table for that size, as 50 ft for 3/4 in, 70 ft for 1 in, etc. If any one section would exceed the limit length, it must be made of larger pipe. Thus, 6 outlets cou[illegible]

* The Denver Gas and Electric Company.

† See, also, Lighting and Illumination of Buildings, pages 1437 to 1456

‡ With the exception of typographical changes made to conform to the rest of the book, these rules are quoted literally. Editor-in-chief.

howing the Correct Sizes of House-Pipes for Different Len Pipes and Number of Outlets

Lengths of pipes in feet								
⅜-in pipe	½-in pipe	¾-in pipe	1-in pipe	1¼-in pipe	1½-in pipe	2-in pipe	2½-in pipe	3-in pipe
20	30	50	70	100	150	200	300	400
....	27	50	70	100	150	200	300	400
....	12	50	70	100	150	200	300	400
....	....	50	70	100	150	200	300	400
....	....	33	70	100	150	200	300	400
....	....	24	70	100	150	200	300	400
....	....	18	70	100	150	200	300	400
....	....	13	50	100	150	200	300	400
....	....	....	44	100	150	200	300	400
....	....	....	35	100	150	200	300	400
....	....	....	30	90	150	200	300	400
....	....	....	25	75	150	200	300	400
....	....	....	21	60	150	200	300	400
....	....	....	18	53	130	200	300	400
....	....	....	16	45	115	200	300	400
....	....	....	14	41	100	200	300	400
....	....	....	12	36	90	200	300	400
....	....	....	....	32	80	200	300	400
....	....	....	....	29	73	200	300	400
....	....	....	....	27	65	200	300	400
....	....	....	....	24	58	200	300	400
....	....	....	....	22	53	200	300	400
....	....	....	....	20	49	200	300	400
....	....	....	....	18	45	190	300	400
....	....	....	....	17	42	175	300	400
....	....	....	....	12	30	120	300	400
...	....	....	....	.....	22	90	270	400
...	....	....	....	.....	17	70	210	400
...	....	....	....	.....	13	55	165	400
...	....	....	....	.....	.....	45	135	330
...	....	....	....	.....	.....	27	80	200
...	....	....	....	.....	.....	20	60	150
...	....	....	....	.....	.....	.....	33	80
...	....	....	....	.....	.....	.....	22	50
...	....	....	....	.....	.....	.....	15	35
...	....	....	....	......	.....	.....	.....	28
...	....	....	....	.....	.....	.....	.....	21
...	....	....	....	.....	.....	.....	.....	14

rough 75 ft of 1-in pipe, but 1¼ in would have to be use uccessive sections work out to the same size of pipe and t exceeds the longest length in the table for that size pipe, t the meter of the next larger size. For example, if we h plied through 45 ft of pipe and these 5 and 5 more, n 30 ft of pipe, we should find by the table that 10 outlet quire 1-in pipe, and that 5 outlets through 45 ft would al as the sum of the two sections, 30 plus 45 equals 75 ft, int of 1 in that may be used in any continuous run, the one nearer the meter, must be made of 1¼-in pipe. Th

tion of the limit in length of any one size in a continuous run may also be shown as follows: Eight outlets will allow of 13 ft of ¾-in pipe in the section between the eighth and ninth outlet (counting from the extremity of the system toward the meter), provided that this 13 ft added to the total length of ¾-in pipe that may have been used between the extremity of the run and the eighth outlet does not exceed 50 ft, which, according to the table, is the greatest length of ¾ in allowable in any one branch of the system. Therefore, up to the eighth outlet, 37 ft of ¾-in pipe could have been used, and yet allow 13 ft of ¾ in to be used in the section between the eighth and ninth outlet. If more than 37 ft had been used, then the entire 13 ft between the eighth and ninth outlets would have to be of 1-in pipe.

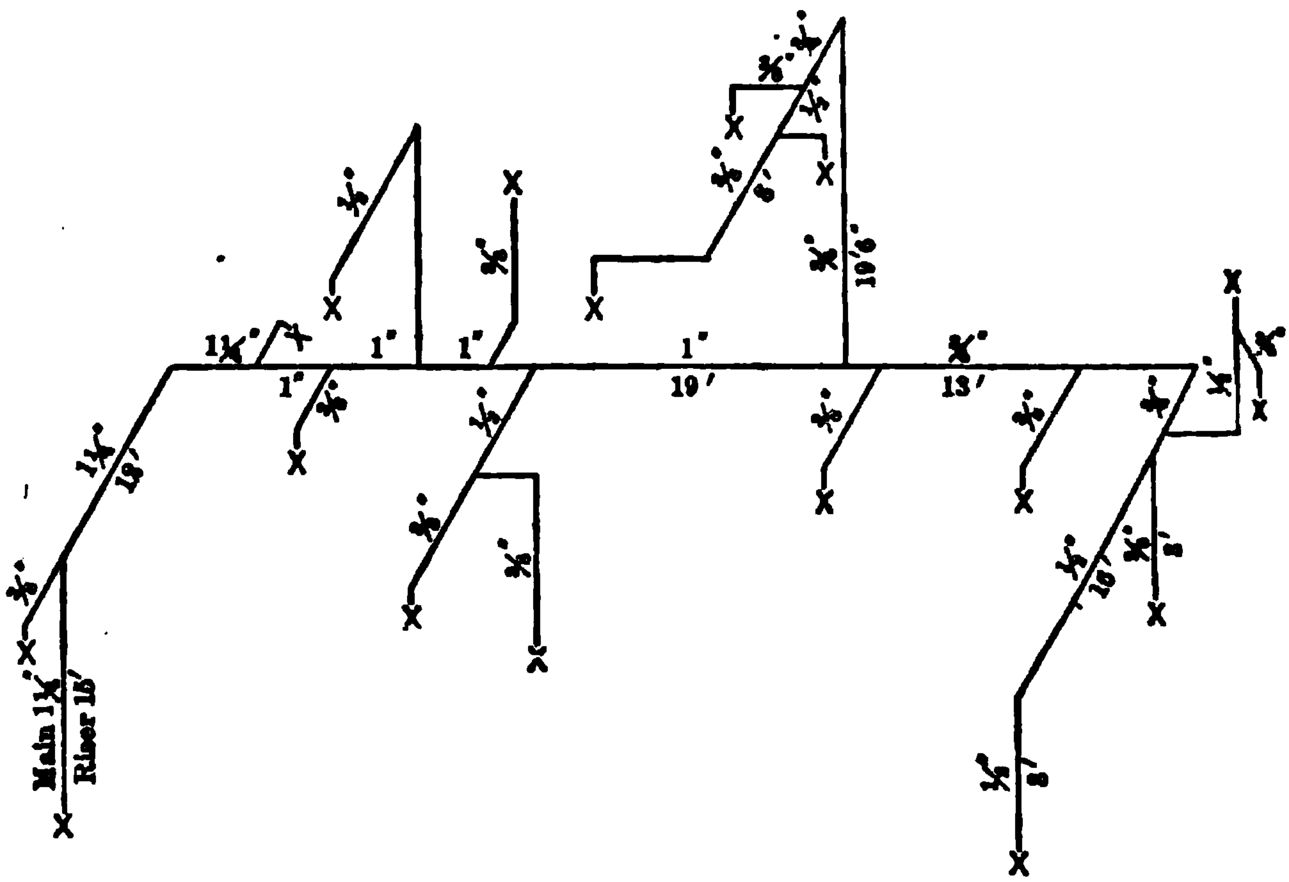


Fig. 18. Diagram of Gas-piping

(11) Never supply gas from a smaller size of pipe to a larger one. If we have 25 outlets to be supplied through 200 ft of pipe, and these 25 and 5 more, making 30 in all, through 100 ft of pipe we should find by the table that 25 outlets through 200 ft would require 2½-in pipe, and 30 outlets through 100 ft would require 2-in piping, but as under this condition a 2-in pipe would be supplying a 2½-in pipe, the 100-ft section must be made 2½ in. The sizes of pipes in Fig 18 are in accordance with the foregoing rules and the table.

LIGHTING AND ILLUMINATION OF BUILDINGS*

By

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General Principles. Objects are illuminated for the sole purpose of making them visible to the eye. The eye, then, is the natural starting-point. When passing upon the merits of any scheme of ordinary illumination, that which should mark it as a success or failure should be the general effect of the scheme upon the eye. Success should be measured largely by the degree of clearness with which the objects are perceived by the eye, as to shape and color. If certain parts of a room or street are too brilliantly lighted, objects in the dimmer portions are not perceived by the eye. If a certain side of one object is too highly illuminated, the general shape of the object is lost, as the eye does not readily perceive its more dimly lighted parts. This is because the eye automatically adjusts itself to the most brilliantly lighted area within its view, and, accordingly, is out of adjustment for perceiving the rest. We should get rid of the idea, therefore, that a light of intense brilliancy is the thing to be sought. It is, in general, highly undesirable. A room may APPEAR brilliantly LIGHTED and yet objects looked at may not be sufficiently well ILLUMINATED for reading or for working purposes. The lights APPEAR brilliant to the eye, but because they throw their strongest rays in other directions than those in which they are needed for use, they do not give efficient illumination.

Distinction between Light and Illumination. There is not only a great difference between LIGHT and ILLUMINATION, but there is a great difference between a brilliantly lighted room and a well-illuminated one. When anybody is asked whether a room is well illuminated or not, the chances are ten to one that he at once looks at the light itself. If the light appears to him to be brilliant and dazzling, he will invariably say, "Why, of course, the room is well lighted." He should first look away from the light at the objects around the room or underneath the light. If these can be seen clearly and easily, then the room is well ILLUMINATED. Afterwards he should look at the lights themselves, and if they appear soft and pleasing to his eyesight the room is well LIGHTED. A room in which the lights appear soft to the eye and yet in which the eye can distinguish objects clearly is both well lighted and well illuminated. A room in which the objects appear clear to the eye while the lights remain dazzling is well illuminated but badly lighted. A room in which the lights appear soft to the eye and the objects not clearly illuminated, is well lighted, but badly illuminated. A room in which the lights appear dazzling to the eye and the surrounding objects or those underneath appear not clear to the eyesight is both badly lighted and badly illuminated. [An] axiom in good artificial illumination is to keep the illumination of objects as [str]ong as is necessary, but the intensity or brilliancy of the lights as low as pos[sib]le. By doing the first we enable the eye to see better; by doing the second we [ena]ble the eye to feel better and suffer less from temporary discomfort or per[ma]nent injury. It is not generally understood that a light which is dazzling and [bril]liant to the eyesight may not be giving as much illumination as another [sou]rce of light which appears soft, or even dim, by comparison. Thus an open [...]light is more dazzling than an enclosed light, but is less efficient in illumi-

* See, also, Illuminating-Gas and Gas-Piping, pages 1431 to 1436.

nating a room. The problem, then, resolves itself into two parts. The first step should be to secure a kind of lamp which will cause objects to appear in their accustomed colors; that is, the colors in which they appear by sunlight. The second is to so distribute the lamps that the several illuminated surfaces receive their share of the light, and yet no bright light is thrown directly into the eyes.

Nature of Light. All space is supposed to be filled with a medium infinitely lighter than air, called ETHER. The sensation of light is experienced when certain wave-motions in this ether are transmitted to the eye. These wave-motions are called LIGHT-WAVES. Light-waves differ from one another in length and violence. The DIFFERENCE IN LENGTH causes a difference in color. Thus short waves may be blue or violet, while longer waves may be red or orange. If we have a source of light which sends out long ether-waves, we may expect a predominance of red and orange light in it. The sunlight contains waves of practically all lengths and thus is composed of all colors. The DIFFERENCE IN VIOLENCE of the waves gives rise to a difference in intensity of the light. When these light-waves strike an object, they are partly reflected and partly absorbed. Substances differ widely as to the percentage of light they absorb and the percentage they reflect. If two objects are illuminated by the same amount of light, the one which absorbs the less light and reflects the more will appear the brighter. Some objects reflect light-waves of a certain length only, and absorb all the rest. It is this property that gives color to objects. Suppose, for instance, that a piece of cloth were receiving light from the sun, all of which it absorbed except the waves of proper length to cause a sensation of green to the eye. The green waves only would then come from the cloth to the eye, all the rest being absorbed, and the cloth would appear green. If it absorbed waves of all lengths, it would appear black, because no light would be reflected from it to the eye. If now the piece of cloth, which absorbs all wave-lengths except that of green, were exposed to a source of light which was emitting all colors EXCEPT GREEN, there being no green waves to be reflected from it, the cloth in this light would appear black. Suppose a piece of cloth absorbed all colors but two, say violet and red. When light having all wave-lengths fell upon it, it would absorb all the waves except violet and red. These two, the cloth would reflect as a mixture and would appear purple. If, however, the source of light contained no violet waves, it could only reflect the red waves and appear red. This light, then, would not cause the cloth to show its normal color. So in choosing an artificial source of light, it is necessary to select one which will send out all wave-lengths, if we wish to have the different objects appear in their normal colors.

Table I. Colors of Light-Sources *

Sun (at zenith)	White (all colors)
Electric arc	Violet-white
Candle	Orange-yellow
Kerosene	Pale orange-yellow
Gas-flame	Pale orange-yellow
Welsbach (gas)	Nearly white to amber, depending upon composition of mantle
Acetylene-flame	Almost white
Carbon, incandescent	Reddish white
Tungsten or Mazda	Yellowish white
Mercury-arc	Blue-green
Moore tube (carbon dioxide)	White

* Compiled by R. F. Pierce, Welsbach Company.

Experiment has shown that no artificial light except the CO_2 Moore tube is even a remote approximation to daylight. The Welsbach white mantle gives a much whiter light than the tungsten-lamp, although neither can be said to approximate daylight.

Light-Intensity or Brilliancy. Candle-Power. The brilliancy of a source of light is stated as its CANDLE-POWER; that is, the number of standard candles to which it is equivalent. Thus an ordinary open gas-flame, consuming 5 cu ft of gas per hour, is equivalent in brilliancy to about 18 candles, and is said to have an intensity of 18 candle-power. Welsbach lamps, consuming 3 cu ft per hour, average about 75 candle-power; that is, they are equivalent to 75 candles. Since no two sources of light have the same amount of luminous surface, it is customary to rate a lamp by the number of candle-power per square inch of its apparent (or projected) surface. Thus an ordinary candle-flame has about ⅙ sq in of area, and its intensity would be rated as 3 candle-power per square inch; that is, the candle-power it would have if its area consisted of exactly 1 sq in. This is often called the INTRINSIC BRILLIANCY of a light-source.*

Table II. Accepted Values of Intrinsic Brilliancy for Various Light-Sources now in Use *

Light-Source	Candle-power per sq in
Moore tube	0.3–1.75
Frosted electric incandescent-lamp	2–5
Candle	3–4
Gas-flame	3–8
Oil-lamp	3–8
Cooper-Hewitt lamp	17
Welsbach gas-mantle	20–50
Acetylene-burner	75–100
Enclosed alternating-current arc-lamp	75–200
Enclosed direct-current arc-lamp	100–500
Incandescent lamps:	
Carbon, 3.5 watts per candle	375
Carbon, 3.1 watts per candle	480
Gem, 2.5 watts per candle	625
Tantalum, 2.0 watts per candle	750
Mazda or tungsten 1.25 watts per candle	875
Mazda or tungsten, 1.0 watt per candle	1 000
Nernst, 1.5 watts per candle	2 200
Sun, on horizon	2 000
Flaming arc-lamp	5 000
Mazda, nitrogen-filled	7 700
Open arc-lamp	10 000–50 000
Open arc-crater	200 000
Sun, 30° above horizon	500 000
Sun, at zenith	600 000

* E. B. Rowe, Holophane Works.

Intensity of Illumination. Foot-Candle. The extent to which a surface is illuminated is measured in FOOT-CANDLES. A surface has 1 foot-candle illumi-

* The total amount of light given out by a light-source is measured in LUMENS. For definition and use of this term see any standard book on illumination.

nation when it is placed, at right-angles to the light-rays, 1 ft away from a light of 1 candle-power intensity. Thus a paper placed 1 ft away from a 16-candle-power incandescent lamp would be illuminated to 16 foot-candles.

Law of Inverse Squares. The farther away from the light the above paper is held the less the illumination. But if it were held 2 ft away, that is, twice as far as stated above, it would not have one-half the illumination. The illumination which an object receives varies inversely as the square of the distance from the source. Thus, in this example the paper would receive one-fourth as much illumination, or 4 foot-candles. If it were held 3 ft away, it would be illuminated by one-ninth of 16, or 1.6 foot-candles.

Rule. To find the intensity of illumination on any surface, at right-angles to light-rays, divide the CANDLE-POWER of the lamp by the SQUARE of the distance in FEET. The result will be FOOT-CANDLE illumination. This is called the LAW OF INVERSE SQUARES. Accordingly, an unshaded 32-candle-power lamp will illuminate a surface facing it squarely and 1 ft away from it with an intensity of 32 foot-candles, but a surface 4 ft away, with only $32/4^2$, or 2 foot-candles.

Candle-Power and Foot-Candle. Careful distinction should be made between CANDLE-POWER and FOOT-CANDLE. CANDLE-POWER is the measure of the intensity of a source of light. The FOOT-CANDLE is the measure of the intensity of illumination of some surface upon which the light falls.

Example 1. What is the illumination on a surface 5 ft from a 32-candle-power lamp?

Solution. $\frac{32}{5 \times 5} = 1.28$ ft-candles.

Example 2. The illumination required on a printed page for easy reading is about 2 foot-candles. (1) How high above a reading-table should a 16-candle-power lamp be hung? (2) A 32-candle-power lamp?

Solution. $\frac{16}{x^2} = 2 \qquad x^2 = 8 \qquad x = \sqrt{8} = 2.83$ ft (1)

$\frac{32}{x^2} = 2 \qquad x^2 = 16 \qquad x = 4$ ft (2)

The Primary Function of a Lighting-Installation is to supply sufficient illumination as required by the character of the work to which the lighted space is devoted. The following table can be used in computing the amount of electric power or of gas necessary to satisfactorily illuminate the various rooms included.

Since the lower efficiencies of the indirect and semiindirect systems are largely compensated by the lower intensities required as compared to direct lighting the same watts per square foot may be allowed in either case, provided the conditions are fairly favorable to the use of the indirect and semiindirect systems namely, light-cream or yellow ceilings. The following table is based upon rooms of average proportions with light-cream, or yellow ceilings and medium walls High, narrow rooms may require about 10% more, and low, wide rooms about 10% less, energy. Similar allowances may be made for dark or light walls respectively.

Three Systems of General Illumination. To secure the proper illumination, as indicated in Table III, there are three general systems.

Table III. Amount of Gas or of Electric Power Required to Illuminate Rooms Used for Various Purposes

Class of service	* Cu ft of gas per sq ft per hour	* Watts per sq ft
Armory or drill-hall	0.02 -0.025	0.5-0.6
Auditorium	0.04 -0.05	1.0-1.3
Barber-shop	0.06 -0.07	1.5-1 7
Church (see Auditorium)	....	....
Drafting-room	0.10 -0.112	2.5-2.8
Factory (general illumination)	0.01 -0.02	2.5-0.5
Hospital (corridor)	0.016-0.02	0.4-0.5
Hospital (operating-room)	0.14 -0.15	3.5-3.9
Hotel (lobby)	0.06 -0.065	1.5-1.6
Hotel (ball-room)	0.05 -0.052	1.2-1.3
Hotel (dining-room)	0.04 -0.045	1.0-1.1
Hotel (restaurant)	0.06 -0.07	1.5-1.7
Hotel (kitchen)	0.05 -0.052	1.2-1.3
Hotel (writing-room, general illumination only)	0.052-0.06	1.3-1.5
Hotel (billiard-room, general illumination only)	0.06 -0.065	1.5-1.6
Hotel (buffet)	0.065-0.072	1.6-1.8
Library (reading-room)	0.055-0.06	1.4-1.5
Library (stacks)	0.012-0.024	0.3-0.6
Office (banking and accounting)	0.06 -0.065	1.5-1.6
Office (general)	0.052-0.06	1.3-1.5
Office (private)	0.05 -0.52	1.2-1.3
Office (stenographic)	0.06 -0.07	1.5-1.7
Residence (bedroom)	0.012	0.3
Residence (dining-room)	0.036-0.04	0.9-1.0
Residence (hall)	0.008	0.2
Residence (living-room)	0.036-0.04	0.9-1.0
Residence (music-room)	0.02 -0.025	0.5-0.6
Residence (kitchen)	0.05 -0.052	1.2-1.3
School (assembly or class-room)	0.04 -0.045	1.0-1.1
School (class-room, business colleges)	0.055-0.06	1.4-1.5
Stores (piano, furniture, haberdashery, dry-goods, automobile, clothing, cigar)	0.06 -0.07	1.5-1.7
Stores (book, shoe, hardware)	0.055-0.06	1.4-1.5
Warehouses	0.012-0.036	0.3-0.9

* These figures are based upon the use of Welsbach reflex lamps and Mazda electric lamps. For Welsbach KINETIC lamps and nitrogen-filled tungsten-lamps (type C Mazda) use about 0.6 the values in the first and second columns, respectively. Data on gas supplied by R. F. Pierce, Welsbach Company.

(1) **Direct Lighting.** A system is designated as DIRECT when more than one-half the light reaches the area to be illuminated by coming directly from the light-source, without being reflected from the ceiling or walls. This includes all systems using lamps with clear, frosted, translucent, or opalescent globes, or reflectors, in which the light is reflected downward. It is the most efficient system, was the first to be used, and is still the most common. The color of the walls or ceiling has less effect in this system than in the others.

(2) **Indirect Lighting.** A system is designated INDIRECT when all the light is thrown first on the ceiling and walls, and reflected from these to the surface to be illuminated. Any system which conceals the source of light by opaque reflectors is thus INDIRECT. Light finish must always be used on the walls and ceiling

with this system. Even then, the efficiency is usually lower than that of a direct system, but the total absence of glare and shadows and the even distribution of light make this a popular scheme in restaurants, show-rooms, etc., where decorative lighting is desired.

(3) **Semiindirect Lighting.** This system throws most of the light to the walls and ceiling, but allows a small percentage to be diffused through the reflector straight to the area to be illuminated. This system is rapidly coming into favor because apparently we have become accustomed to looking for the source of light and miss it when it is concealed as in the indirect system. The totally indirect fixtures often show up rather unpleasantly as a dark spot against a light background.* This is avoided in the semiindirect system. The slightly higher efficiency of this system is another advantage over the indirect. Any given room may usually be lighted by any one of the three systems although it is generally true that conditions are such as to make one of the three more desirable than either of the other two. The following paragraphs show in detail how each system may be worked out for a given room.

General Considerations † in Direct Lighting

Outlets and Lamps. Outlets should be located in the centers of as nearly as possible square and equal areas into which the ceiling, for the purpose of calculation, may be subdivided. The greater the number of outlets the more uniform the illumination and the greater the freedom from annoying shadows. Unless great care is used in planning the directions in which the light is received by illuminated surfaces, a disagreeable glare from glazed paper is likely to be present. The greater the height of lamps above the illuminated area, the more uniform the illumination. Figures suggestive of good practice in selection of mounting-heights are given in Table IV, page 1444.

General Considerations in Indirect and Semiindirect Lighting

Outlets and Lamps. The location of outlets should in general conform to the requirements for direct lighting, that is, at the centers of approximately square and equal areas. Since glare from glazed papers is minimized when most of the light is received from ceiling-reflection, larger and fewer units are permissible than in the case of direct lighting. The nearer to the ceiling the lamps are placed, the less uniform the illumination and, within reasonable limits, the higher the illuminating efficiency of the installation. Generally speaking, lamps should not be placed less than 2 ft from the ceiling. Aside from this, the position of a fixture should be determined by artistic considerations and reflectors selected which will direct most of the light upon the ceiling without concentrating it enough to illuminate the ceiling unevenly.

A. The Interior Colorings and Finishes.‡ (1) Ceilings especially should be of nearly white, cream, or light-buff colors to efficiently diffuse the light downward. Dark greens, reds, or blues are not advisable since the reduction in illumination caused by a green color, over a cream tint, may easily be from 30% to 60%. On the other hand, this system shows very plainly all dirt and discolorations on the ceiling, and no colors should be used that are so light as to easily show dirt, where there may not be careful cleaning.

* This unpleasant effect can sometimes be avoided by illuminating the underside of the fixture.

† By R. F. Pierce, Welsbach Company and G. S. Fobes, Macbeth-Evans Company.

‡ By G. S. Fobes, Macbeth-Evans Company.

(2) Finishes preferably should be matt, or satin, rather than glazed or varnished. From the matt ceiling-surface the maximum light will always be downward, but the varnished ceiling will reflect specularly, directing light sidewise or showing lamp-images and glare.

(3) Tints and details of decoration should be considered together with the lighting-system, so that daylight-colors and reliefs will not be reversed or distorted by colored light from artificial illuminants and shadows.

B. The Positions of Outlets and of Fixtures. (1) Semiindirect units should, if possible, be placed above the places where maximum light is wanted.

(2) Fixtures should not be so close to side walls as to cause light-spots running down across picture-moldings, etc.

(3) Outlets should be placed logically with reference to the ceiling-panels, so that the more brightly illuminated ceiling-areas will be the ones that on account of their tints, shapes, or decorations, will naturally bear emphasis. If the panels are deep (deep beams), and one outlet is in each panel, it will ordinarily be located at the center. If several panels intervene between units, the fixtures should be on the beams rather than in the panels, to prevent dark ceiling-areas in the shadows of the beams.

(4) Spacing should be such as to have the illuminated ceiling-areas overlap if the ceiling-surface is uniform.

C. The Proper Lamp and Bowl-Sizes. (1) Ordinarily the symmetrical appearance of fixtures with respect to the other interior furnishings will largely determine their sizes, although the bowls should never be so small as not to completely conceal and nearly surround the lamp-bulb.

(2) The smaller the bowl and the brighter the lamp, the less effective the semiindirect system becomes, and the more the effect approaches direct lighting.

D. Shapes and Styles of Bowls. (1) Bowls used close together or hung far from the ceiling should be of the focusing (upward) distribution, while broadly distributing bowls are better when used singly, or when fairly wide apart and close to the ceiling.

(2) Bowls too flat in shape may waste considerable light sidewise to the upper walls and therefore be inefficient.

(3) Wide open-top bowls should not be used in halls, etc., where the bare lamps are visible to the observer from above, nor on or below the level of a balcony or mezzanine.

E. Care of Fixtures. (1) The average saving in light (expressed in terms of cost of current) that results from washing once and dusting once monthly, will be from four to ten times the cost of such cleaning. Bowls often collect films of dust which are not visible and which materially reduce the efficiency both of reflection and transmission.

(2) A bowl with a dust-cap, button-ornament, or small area of thick glass at the bottom, will allow dead insects or dirt to collect at that point without marring the appearance of the unit.

(3) Dilute ammonia is an excellent glass-cleanser.

(4) Fixtures should be arranged to be lowered, for cleaning, from above if on a very high ceiling in a church or similar structure.

(5) It should be possible to easily raise the lamp or lamps from within the bowl, to allow of dusting or wiping out.

Figures suggestive of good practice in the selection of mounting-heights and types of light-distribution are given in Table VI.

Table IV. Direct System

LAMP-SIZE, MOUNTING-HEIGHT AND SPACING*

Mounting-height	Commercial size of lamps in watts = W and cubic feet per hour		Watts per square foot = w and cubic feet per square foot		Ideal spacing = distance $\sqrt{\frac{W}{w}}$		Minimum spacing-distance		Maximum spacing-distance	
ft	Watts	cu ft	Watts	cu ft	ft	in	ft	in	ft	in
7 to 10	40	1.6	0.5	0.02	9	0	8	0	10	0
			1.5	0.06	5	2	4	6	6	0
			2.5	0.10	4	0	3	9	4	3
8 to 13	60	3.0	0.5	0.02	11	0	9	6	12	9
			1.5	0.06	6	4	5	6	7	3
			2.5	0.10	4	11	4	6	5	6
12 to 16	100	4.0	0.5	0.02	14	5	12	6	16	0
			1.5	0.06	8	2	7	0	9	6
			2.5	0.10	6	4	5	8	7	0
14 to 20	150	6.0	0.5	0.02	17	4	15	0	20	0
			1.5	0.06	10	0	9	0	11	0
			2.5	0.10	7	9	7	0	8	6
17 to 27	250	10.0	0.5	0.02	22	5	20	0	25	0
			1.5	0.06	12	11	11	9	14	3
			2.5	0.10	10	0	9	0	11	0
25 to 35	400	16.0	0.5	0.02	28	2	25	0	31	6
			1.5	0.06	16	4	15	0	17	9
			2.5	0.10	12	7	11	6	13	6
30 to 40	500	20.0	0.5	0.02	31	7	28	0	35	6
			1.5	0.06	18	6	16	6	20	9
			2.5	0.10	14	2	12	6	15	0

* To determine the size of equivalent Welsbach lamps allow 1 cu ft per hour for each 25 watts. Adapted from the Electric Journal, by A. J. Airston.

The Designing of General Illumination by Each System Using Tungsten or Welsbach Lamps

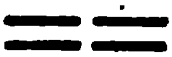
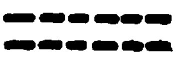
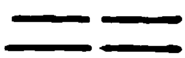
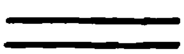
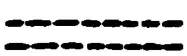
(1) From Table III should be determined the watts per square foot desirable for the given class of work, and the total number of watts necessary should then be computed.

(2) From Table IV should be obtained the size of unit desirable for a given height of room and the number and spacing of fixtures then computed.

(3) The ceiling should be laid off in squares the sides of which are as nearly as possible equal to the value of the ideal spacing. One fixture should be located at the center of each square.

(4) Each lamp should be checked up on the plan to see that it is useful and clear of obstacles, and the layout incorporated into the building plans using the standard methods and symbols for electricity or gas as the case may be.

Table V. Standard Symbols for Gas-Piping Plans*

 4	Ceiling-outlet; gas only. Numeral indicates the number of single-mantle gas-lamps.
	Single-lamp outlet (ceiling-units, pendants, etc.); gas only.
 $\frac{4}{2}$	Ceiling-outlet; combination. $\frac{4}{2}$ indicates 4 electric lamps and 2 single-mantle gas-lamps.
 2	Bracket-outlet; gas only. Numeral indicates the number of gas-lamps.
 $\frac{4}{2}$	Bracket-outlet; combination. $\frac{4}{2}$ indicates 4 electric lamps and 2 gas-lamps.
 2	Baseboard-outlet; gas only. Numeral indicates number of gas-lamps.
	Floor-outlet; gas only.
	Special outlet (for portable lamp, heater, etc.); gas only.
 $\frac{2}{5}$	Outlet for outdoor-standard or pedestal; gas only. $\frac{2}{5}$ indicates 2 gas-lamps, with 5 mantles per lamp.
 $\frac{6}{2/5}$	Outlet for outdoor standard or pedestal; combination. $\frac{6}{2/5}$ indicates 6 electric lamps, and 2 gas-lamps, with 5 mantles per lamp.
 4	Arc-lamp outlet; gas only. Numeral indicates the number of mantles.
 2	Pump or pneumatic lighting-system. Numeral indicates the number of lamps to be operated from one pump.
2 	Push-button for magnet-valve. The numeral indicates the number of lamps to be operated from one push-button switch.
	Meter-outlet.
	Main or supply-pipe concealed under floor.
	Main or supply-pipe concealed under floor above.
	Main or supply-pipe exposed.
	Branch pipe concealed under floor.
	Branch pipe concealed under floor above.
	Branch pipe exposed.
	Street gas-main.
	Battery-outlet.
	Riser.

* Illuminating Engineering Laboratories, Welsbach Company.

Distance from Floor to Center of Wall-Outlets*

	ft	in
Living-room	5	6
Chambers	5	0
Offices	6	0
Corridors	6	3
Push-button switches or pneumatic pumps	4	0

* Illuminating Engineering Laboratories, Welsbach Company.

Examples of Design of Lighting-System for accounting-office, 63 by 25 ft with 13-ft ceiling (Fig. 1). Walls and ceiling-light in color.

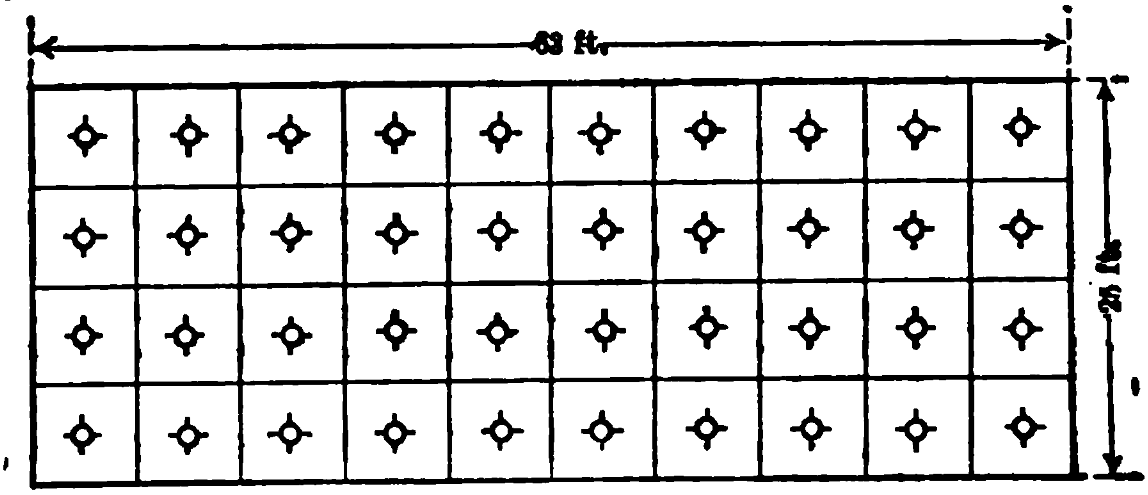


Fig. 1. Plan of Ceiling-lights

Direct System

Watts per sq ft	= 1.5 (Table III)
Total watts	= 1.5 × 63 × 25 = 2 400 (nearly)
Unit, size of	= 60 watt electric (Table IV)
	= 3 cu ft per hour, ordinary inclosed gas, or } about
	= 2 cu ft per hour, Welsbach KINETIC } about
Number of units	= 2 400/60 = forty
Spacing (average) desired	= 6 ft 4 in (Table IV)
Number of rows	= 25/6⅓ = four
Number of outlets per row	= 40/4 = ten
Spacing between rows	= 25/4 = 6¼ ft
Spacing in rows	= 63/10 = 6⅓ ft
Spacing-average	= 6 ft 4 in.

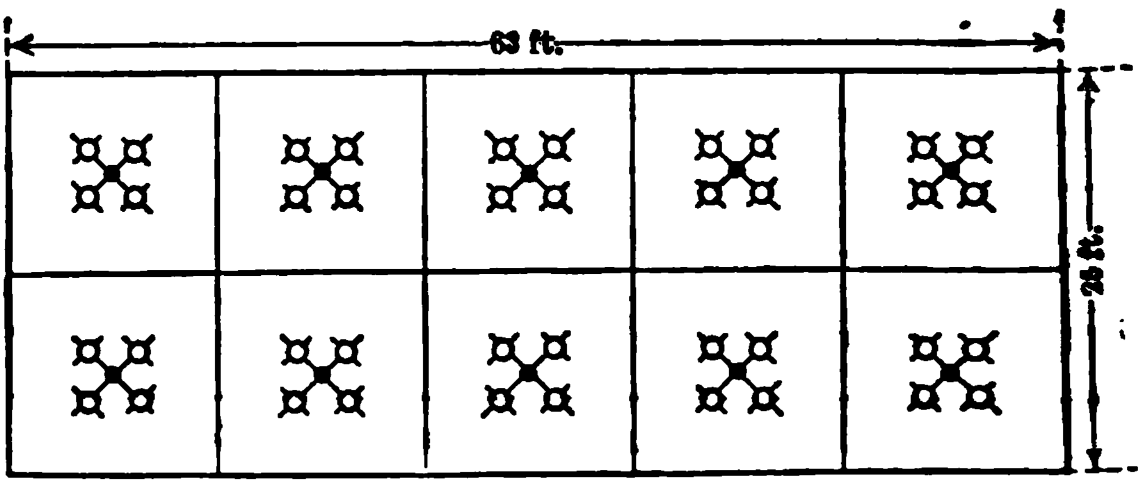


Fig. 1A. Modification of Plan Shown in Fig. 1

Fig. 1A is a modification of the plan shown in Fig. 1, and is a great improvement. It will not produce such even illumination but will result in a much

rtistic effect, especially if fixtures are chosen which harmonize with the furnishings of the room. The lamps are placed in groups of four on ten fixtures and these are equally spaced throughout the room. Here again it is always possible to use lamps of higher wattage at any point where the illumination is not sufficient. The importance of a proper choice of reflector is shown from a study of Figs. 2 to 5.* It will be noted in Fig. 2 how a bare tungsten-lamp throws the greater part of its light to the walls. The distribution of any light can be controlled to a remarkable extent by the use

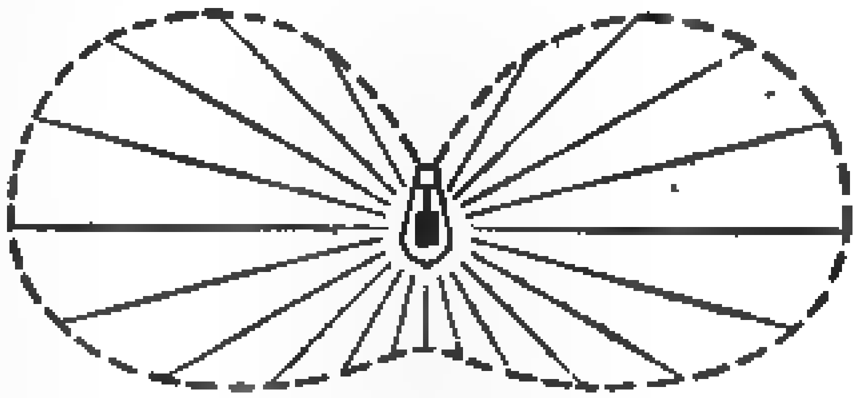
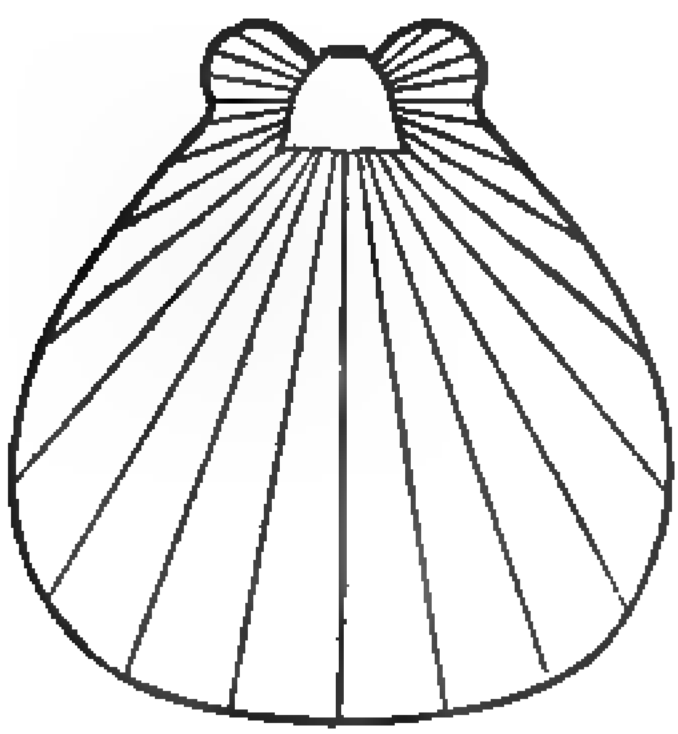


Fig. 2. Distribution of Candle-power about a Bare Tungsten Lamp



. 3. Holophane Reflector. Extensive Distribution of Light

Fig. 4. Holophane Reflector. Intensive Distribution of Light

the proper reflector. Figs. 3 to 5 show how the several types of Holophane lectors distribute the light.

* Furnished by E. B. Rowe, of the Holophane Works.

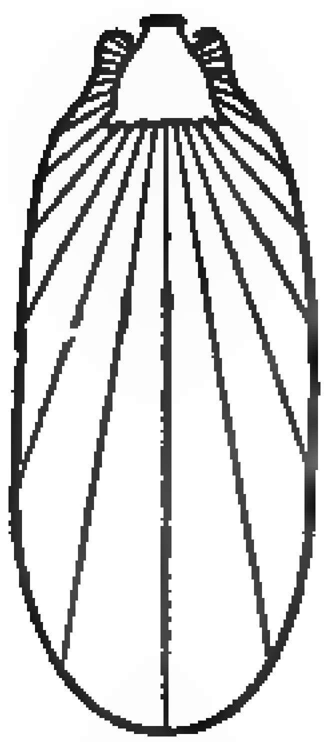


Fig. 5. Holophane Reflector. Focusing Effect on Light

Fig. 6. Example of Type of Fixture Used in Semiindirect System. Macbeth-Evans Company

Indirect or Semiindirect Systems, for Electricity

Watts per sq ft	= 1.5 (Table III)
Total watts	= 2 400, nearly
Average spacing	= 14 to 24 ft (Table VI)*
Select 25/2	= 14 ft for lamps in two rows
Number of units in row	= 63/12.5 = five
Spacing in row	= 63/5 = 12 ft 7 in, about
Type of reflector	= Concentrating (Table VI)
Distance from reflector to ceiling	= 30 in (Table VI)
Number of units	= two rows of five each = ten
Watts per unit	= 2 400/10 = 240
Lamps per unit	= one, 250 watts four, 60 watts six, 40 watts

Calculations for this Example for Gas-Lighting

Welsbach kinetic burner used.

Cu ft per hour per sq ft	= 0.06 × 0.6 = 0.036 (Table III)
Total hourly consumption	= 63 × 25 × 0.036 = 57 cu ft per hour.
Average spacing (see above)	= 12½ ft
Number of units	= ten

Consumption per unit = 57/10 = 6

Reflector and mounting-height as in preceding problem

Lamps per unit = one, 6 cu ft

Lamps per unit = two, 3 cu ft, etc.

* See How to Use Table VI, immediately following the Table.

Table VI. For Determining Number of Ceiling-Outlets, Type of Reflec and the Distance from Top of Reflector to Ceiling for Indirect and Semiindirect Lighting *

Height of ceiling in feet		10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
20	Distributing						...	...	...	...		...	48	[illegible]	48	5′-0	6′-6
	Concentrating						...	...	48	5′-0	6′-6	7′-0	...	...	...	...	...
19	Distributing						...	...	...	...	...		48	48	5′-0	6′-0	...
	Concentrating						...		48	6′-0	6′-6	6′-6	...		...	...	...
18	Distributing						...	...			...	42	[illegible]	[illegible]	6′-0	...	...
	Concentrating						...	42	5′-0	6′-0	6′-0	...	...	...	...	...	...
17	Distributing						...	...	...	...	42	42	4′-6	6′-0	...	...	...
	Concentrating						36	4′-6	6′-0	6′-0	...	...	...		...	...	...
16	Distributing	...	...	...	...		...		...	42	42	4′-6	5′-6				
	Concentrating	...	...	...	...	36	42	5′-6	5′-6	...	...	...	...				
15	Distributing	...	...	...	...	...	...		30	[illegible]	4′-6	5′-0	...				
	Concentrating	...	...	...	30	[illegible]	4′-6	5′-0	...	...	...	...	...				
14	Distributing	...	...	...	...	...	...	30	30	[illegible]	5′-0	...	...				
	Concentrating	...	...	...	30	48	5′-0	...	...	...	...	...	...				
13	Distributing	...	...	...	...	...	30	30	48	...	...	...	...				
	Concentrating	...	...	30	36	48	...	...	...	...	...	...	...				
12	Distributing	...	...	...	...	24	30	42	48	...	...	...	...				
	Concentrating	...	...	30	48	...	...	...	...	...	...	...	...				
11½	Distributing	...	...		24	[illegible]	36	[illegible]	...	...	...	...	...				
	Concentrating	...	24	36	...	...	...	...	...	...	...	...	...				
11	Distributing	...	...	...	24	30	42	48	...	...	...	...	...				
	Concentrating	...	24	42	...			...	...	...	...	...	...				
10½	Distributing	...	...	...	24	36	42	...	...	...	...	...	...				
	Concentrating	...	30	42	...	...	...	...	...	...	...	...	...				
10	Distributing	...	...	18	24	36	...	...	...	...	...	...	...				
	Concentrating	18	30		...	...	...	...	...	...	...	...	...				
9½	Distributing	...	18	18	30	36	...	...	...	...	...	...	...				
	Concentrating	18	...	...	...	...	...	...	...	...	...	...	...				
9	Distributing	...	18	24	30	...	...	...	...	...	...	...	...				
	Concentrating	...	...	...	...	...	...	...	...	...	...	...	...				
8½	Distributing	...	18	24	...	...	...	...	...	...	...	...	...				
	Concentrating	...	...	...	...	...	...	...	...	...	...	...	...				
8	Distributing	12	18	...	...	...	...	...	...	...	...	...	...				
	Concentrating	...	...	...	...	...	...	...	...	...	...	...	...				

One side of the limiting square in feet that can be uniformly illuminated from one center outlet

Table VI gives the distance in inches (except as noted) from the top of the refle the ceiling to obtain the desired distribution of light from one ceiling-outlet.

Where values are not given to the left, it is advisable to submit data to illumi engineers and for greater ceiling-heights than 20 ft.

* H. B. Wheeler, X-Ray Eye-Comfort Company.

An idea of the appearance of some of the typical modern fixtures using gas or electricity in these systems of lighting may be obtained from Figs. 6, 7 and 8.

Fig. 7. Type of Fixture in Indirect Illumination. National X-Ray Reflector Company

Fig. 8. Fixture Used for Gas by Either Indirect or Semiindirect System

How to Use Table VI. In the first column of Table VI is located the height of ceiling, in this case, 13 ft. The last square to the right of this figure, which has a number in it, is noted. In this case the last square to the right of 13 ft, which has a number in it, contains the number 48. By following this column containing the number 48 down to the figures printed in heavy type at the bottom of the table, the heavy-faced number in this case is found to be 24. This 24 is the length of the side of the largest square which a single fixture can properly illuminate when the ceiling is 13 ft high. The 48 which is opposite the 13 is merely the number of inches the fixture must be hung from the ceiling. Thus the largest squares into which we can possibly divide the ceiling have 24-ft sides. But a room 25 ft wide cannot be divided into 24-ft squares. We are compelled, therefore, to divide it into squares of a smaller size, since the fixtures will not illuminate any larger square. The greatest length into which we can divide 25 ft is 12½ ft. We may, then, decide to use fixtures which will illuminate 14-ft squares. Locate the number 14 in heavy type at bottom of table, and trace up the column in which it is found until the square is reached which is opposite the ceiling-height of 13 ft. Here the number 30 is found. This means we must hang the fixture so that the top of it is 30 in from the ceiling in order to get the desired results. Looking along the squares to the right of the one in which we find the 30, we find the word CONCENTRATING which signifies the type of reflector advised for this installation.

School-Room Lighting.* The following illumination-constants have been worked out by experiments and experience covering a wide range of conditions. In each of the following cases light tinted walls and ceilings are taken as a standard.

Auditoriums and Lecture-Halls. Since no continuous reading is required here, 0.75 watt per sq ft, direct system, is all that is needed, if it is properly diffused to a pleasant softness.

Class-Rooms and Laboratories. These must be lighted for the purpose of writing notes and taking accurate readings of instruments. Thus 1¼ watts per sq ft, direct system, are required.

Wood-Working Shops. The surfaces here are generally high and offer good reflecting properties, so that 1½ watts per sq ft, direct system, are sufficient.

Machine-Shops. Because the belts, machines and dingy floors offer great absorbing surfaces at least 2 watts per sq ft, direct system, are necessary.

Foundries. The dark molding-sand and the dust and smoke in the air make 3 watts per sq ft, direct system, necessary.

Drafting-Rooms. The semiindirect system with 2¼ watts per sq ft (about the equivalent of 1¾ watts per sq ft, direct system) has proved highly satisfactory.

Illumination by Gas.† Recent progress in incandescent gas-lighting has resulted in the development of appliances in which practically all of the shortcomings of previous types are overcome, and except for inaccessible locations, or where lamps are very infrequently lighted, there is little to choose between the two illuminants, gas and electricity, upon the score of convenience or of artistic possibilities, while the greater economy of gas-lighting (often in the ratio of about 2½ to 1) coupled with the freedom from interruption which characterizes gas-service makes it desirable to pipe all buildings, particularly residences, for gas throughout, preferably installing combination-fixtures and providing wall-outlets and baseboard-outlets for the connection of the various gas-operated conveniences which are being developed in rapidly increasing numbers.

Welsbach Kinetic-Burner Lamps With Nearest Equivalent Sizes in Electric Incandescent Lamps

Lamps	Mazda watts	Nitrogen-filled Mazda (Type C) watts
1 mantle 2.5 cu ft per hour	two, 40	....
2 mantles 5.0 cu ft per hour	150	....
3 mantles 7.5 cu ft per hour	250	....
4 mantles 10.0 cu ft per hour	six, 40	....
5 mantles 12.5 cu ft per hour	400	....
6 mantles 15.0 cu ft per hour	500	....
8 mantles 20.0 cu ft per hour	....	500
10 mantles 25.0 cu ft per hour	....	750

Gas-Lamps are available in a variety of types and sizes. The most recent development is the KINETIC burner of the Welsbach Company in which the efficiency is increased by from 50% to 100% over the previous types. With this burner no enclosing glassware or housings are required, and the lamp is said

* A. L. Williston. † R. F. Pierce, Welsbach Company.

to require no attention beyond the renewal of mantles every 2 000 burning-hours. There is practically no depreciation in candle-power during this interval. Ignition is accomplished either by a pilot-flame burning about 1/10 cu ft per hour, or by electrical means, and several types of distant control are available. The following table gives the sizes in which these lamps may be obtained and the nearest equivalent sizes in electric incandescent lamps.

Selection of Illuminants

(1) Factors favorable to the use of electricity:
- Units less than 60 candle-power required.
- Lamps in inaccessible positions.
- Lamps lighted at infrequent intervals.
- Lamps placed very close to ceiling (12 in or less).
- Poor gas service as regards:
  - Pressure-regulation (more than 50% variation from minimum).
  - Non-uniformity of gas-quality.
  - Imperfect purification.
- Good electric service as regards:
  - Voltage-regulation.
  - Freedom from liability to derangement by accident.
- Non-rigid fixtures.

(2) Factors favorable to the use of gas:
- Units of 60 candle-power or more.
- Accessible locations.
- Frequent use of lamps.
- Lamps placed 15 in or more from ceiling.
- Good gas service as regards:
  - Pressure-regulation (not more than 50% variation from minimum).
  - Uniformity of gas-quality (chemical composition).
  - Proper purification.
- Poor electric service as regards:
  - Voltage-regulation (more than 5% variation from maximum) most likely on alternating-current circuits.
  - Liability to derangement by accident (overhead circuits).
  - Rigid fixtures.

Hygiene.* From the hygienic point of view there is little to choose between the two illuminants. The investigations of Dr. Rideal have shown that: (1) Gas-light positively improves the air for breathing purposes under the actual conditions of use. The causes of this improvement are the acceleration of ventilation, the destruction of disease-germs and the addition of necessary moisture. Gas-burners give rise to stronger air-currents and invariably produce a more active ventilation and diffusion of air than electric lights; hence, along with the products of the gas-burner, the exhalations of persons present are more rapidly removed; (2) The ascending currents of air from the gas-lights on reaching the ceilings rapidly part with their heat, which is conducted away by the rafters and joists; (3) The electric lamps produce more heat than is commonly accredited to them, and this is the explanation of the unexpected result that the average temperature of the room is practically the same under either illuminant, and that the electric light does not show the superiority in coolness usually claimed. When excessive temperatures are encountered in gas-lighted

* See Relative Hygienic Values of Gas and Electric Lighting, by Samuel Rideal, Transactions Royal Sanitary Institute, March, 1908.

rooms, it will be found due to the radiant heat from low-hung lamps of excessive size. On account of the economy of gas-lighting, it is a common practice to provide from four to six times as much illumination as is required. Dr. Rideal's tests also emphasized, what is a matter of common experience, that under direct lighting, the lower brilliancy of the gas-mantle reduced the glare from glazed papers to such an extent as to be noticeable in the results: "The sensitiveness of the eye to light as measured in the perception-test diminished very markedly after exposure to the electric light, while no corresponding effect is noticeable after the eye has been subjected to gaslight. All the results point strongly in the same direction, namely, that gaslight, as used in these experiments, is less fatiguing to the eye than electric light." Under semi-indirect or indirect lighting, of course, no such disparity in effect is found.

The Foregoing Rules Indicate the General Practice in planning the illumination of a room. It must be said, however, that this set of rules must not be followed too slavishly. In illumination no rules can take the place of judgment and intelligence. Each project must be considered more or less as a problem by itself, for which previous experience and former installations should be made to furnish data and to suggest methods. It is well, therefore, when planning the illumination of a room, to visit as many similar rooms as possible, note the effect of the systems in use and obtain data as to their efficiency, cost, etc. The most successful scheme may then be used as the basis for planning the desired installation.

The Diffusion of Light through Windows *

Tests on the Diffusion of Light by Glass. Abstracts from report of Charles L. Norton, on an elaborate series of tests made at the Massachusetts Institute of Technology:† The results of the tests on a score or more of different glasses may be stated briefly. We may increase the light in a room 30 ft or more deep to from three to fifteen times its present effect by using FACTORY-RIBBED GLASS instead of PLANE GLASS in the upper sashes. By using prisms we may, under certain conditions, increase the effective light to fifty times its present strength. The gain in effective light on substituting ribbed glass or prisms for plane glass is much greater when the sky-angle is small, as in the case of windows opening upon light-shafts or narrow alleys. The increase in the strength of the light directly opposite a window in which ribbed glass or prisms have been substituted for plane glass is at times such as to light a desk or table 50 ft from the window better than one 20 ft from the window had previously been lighted.

The Kinds of Glass Tested were as follows:

(1) Ground glass of different degrees of fineness.

(2) Rough plate or hammered glass.

(3) Ribbed or corrugated glass, with five, and eleven and twenty-one ribs to the inch, the corrugations being sinusoidal in outline, as in *A*, Fig. 9, and the back of the plate smooth.

(4) Glass known as MAZE, FLORENTINE or FIGURED, in which a raised pattern is worked upon one side, practically roughening the whole surface.

(5) Wash-board glass, corrugated, with twenty-one ribs to the inch on one side and five ribs to the inch on the other side, the ribs being parallel.

(6) Skylight-glass, which has five ribs to the inch on each side, the groove on one side being opposite the rib on the other, giving a sinuous section *B*, Fig. 9.

* See, also, the subjects Pressed Prism-Plate Glass and Prism Glass, Part III, pages 1577 to 1579.

† From Report No. III, Insurance Engineering Experiment Station, September, 1902.

(7) Ripple-glass, with rippled surfaces on both sides; of very beautiful appearance and a clear white color.

(8) Glass ribbed on one side and figured on the other.

(9) Ribbed glass with a wire net pressed into it, to increase its resistance to fire.

Of these several specimens, one or two may be dismissed with brief mention. Ground glass is of little value, except as a softening medium for bright sunlight. Its rapidly increasing opaqueness with moisture and dust makes it undesirable as a window-glass. The common rough plate has very little action as a diffusing-medium, giving no perceptible change in the effective light. Ripple-glass has great value as a diffusing-medium in small rooms with nearly open horizon. Of the ribbed glasses, the fine Factory-Ribbed, with twenty-one ribs to the inch, is distinctly the best, not in all probability because of the fineness, but because of the greater sharpness of the corrugations.

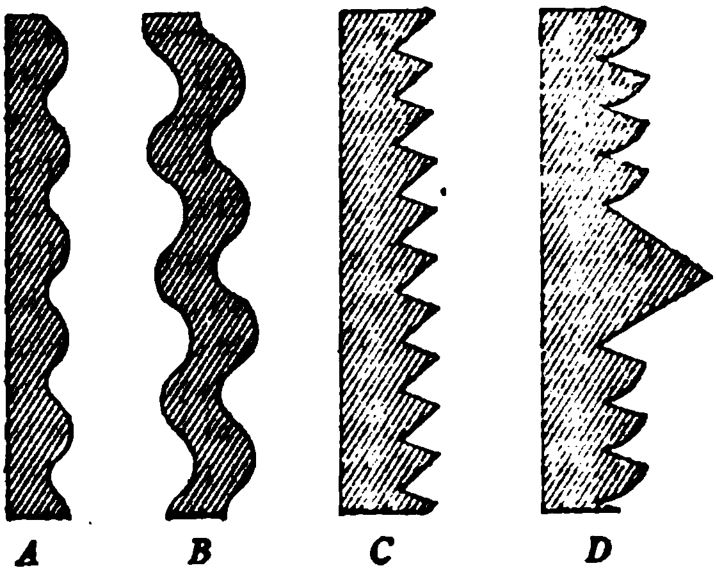


Fig. 9. Types of Ribbed or Prism-glass

The Ribbed wire-glass is about 20% less effective than the ordinary Factory-Ribbed glass. The addition of a second corrugation upon the back of the plate giving the Skylight and Wash-Board glass is of no apparent value. The raised pattern imprinted upon one surface of the glass, as in the case of the Maze glass, gives the widest diffusion, especially in bright sunlight. A raised figure, when worked upon the back of the Ribbed glass, renders it less offensive to the eye in bright sunlight, but less effective in deep rooms. The only glasses of this group which it is worth while, then, to discuss further are the Factory-Ribbed and the Maze glass.

The second group comprises the following glasses:

(1) The Luxfer prisms.

(2) The Solar prisms.

(3) The Daylight-prisms.

(4) The glass of prismatic section made by the Mississippi Glass Company.

(5) Three-way prisms.

(6) Maltby prisms.

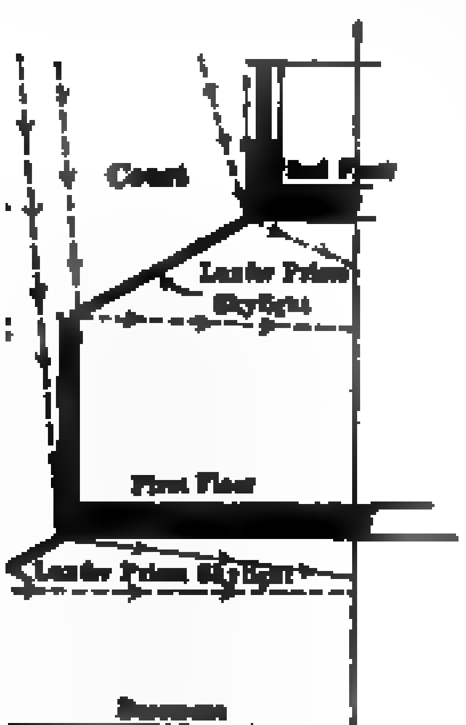
The Luxfer prism consists of a plate smooth on one side and deeply notched on the other as in *C*, Fig. 9, the teeth or prisms being of very flat, smooth faces of brilliant appearance. The glass is clear white, and the prisms used in canopies and in the major part of the vertical glazing are made in tiles or plates about 4 in square. Tiles are built up in large sheets in frames of copper or brass, so made as to give to the sheets of tiles a strength and durability far in excess of a single sheet of the same size. The Luxfer prisms are made for factory-use in large sheets, as well as in the small tiles. The Solar prisms are made in small tiles, which are held together in a metal frame to make large sheets. The main difference between the Solar and Luxfer prisms is that the under face of the former prism is curved instead of plane, as in *D*, Fig. 9. The Daylight-prisms tested were made in large sheets and of approximately the same cross-section and general appearance as the Luxfer prisms for factory-use. No tiles of Daylight-prisms were tested, as none came to hand in time for the test. The Mississippi prism glass is much like the other prisms in cross-section, but the ridges or

ns do not run across the plate in a straight line, but in a wavy or sinuous line. advantage arising from this over the straight-edge prism was detected.

onclusions. (1) The conditions in a room less than 15 ft deep are such , except with a skylight of less than 45°, it is not advisable to alter the general se of the light by using a prismatic or ribbed glass. A nearly hemispherical sion, such as is given by the e or Ripple-glass, is ordiy preferable.

) When a room is from 20 ft deep, or even more, and a skylight of 60° or less, the d and prismatic glass results very great gain in effective . The gain in brilliancy is as to make a basement with canopies as light as a d story with plane glass. oms with windows opening light-shafts and narrow s with very limited openings e sky, where the available is now small, may have the light 20 ft back from the window increased ten enty times by using prisms; and, by using canopies of prisms, it is somepossible to strengthen the light fifty to one hundred times. With skys of 30°, or less, and in deep rooms, the relative efficiency of the prism creases greatly. The refraction of the incident ray in a case of the ribbed glass and prism is shown in Fig. 10. Ribbed and maze glass are of very great value in softening the light, especially in the case of such windows as are exposed to the direct sun, aside from their effectiveness in strengthening the light at distant points. With the Maze glass, the artist may have, in all weather and in all directions, what is in effect a much-desired NORTH LIGHT. The photographer may have in this way as well diffused a light as he now has with cloth screens or shades, and with a much greater intensity. To be efficient in rooms 20 ft deep or more, ribbed glass should be set with its ribs horizontal, and where the sunlight falls upon it, it should be provided with thin white shades. All inferences drawn from the test are made upon the assumption that the windows are to be glazed with diffusing glass only in the upper half, which is the common practice. If the lower sash is to be glazed diffusing glass as well, a further increase of about 25% may be ed.

Fig. 10. Refraction of Light in Ribbed and Prism-glass



1. Basement and First Story Lighted from Court

nsidering both expense and efficiency, the following general suggestions are Use Maze or Ripple-glass in small rooms or offices not more than 15 or 20 ft use Factory-Ribbed glass in rooms from 30 to 50 ft deep, with sky-angles of more; use prisms or Factory-Ribbed glass, in sheets, in all vertical win-

dows in rooms more than from 50 to 60 ft deep, with sky-angle of less than 45°. With a sky-angle of less than 30° use prisms in canopies. Fig. 11 shows an effective method of lighting the basement and first story where the light must come from a court.

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ELECTRIC WORK FOR BUILDINGS

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General Considerations and Definitions. Electrical energy is now in common use, furnishing power, heat and light, operating bells and buzzers, and transmitting messages by telephone and telegraph. In order to accomplish these results, a current of electricity must flow around an electric circuit. The nature of electricity is not known, but the flow of it through an electric circuit is analogous to the flow of water through a system of pipes.

Current. Amperes. The flow of water is measured in GALLONS PER SECOND. The flow of electricity is measured in AMPERES. An ampere-flow of electricity is analogous to a gallon-per-second flow of water. The amperes thus indicate the quantity of electricity flowing through an electrical appliance in one second. About ½ ampere is flowing through an ordinary carbon-filament incandescent lamp when it is glowing at 16 candle-power. The same current of ½ ampere causes a modern tungsten lamp to produce over 40 candle-power. An arc-lamp usually requires a flow of from 5 to 10 amperes.

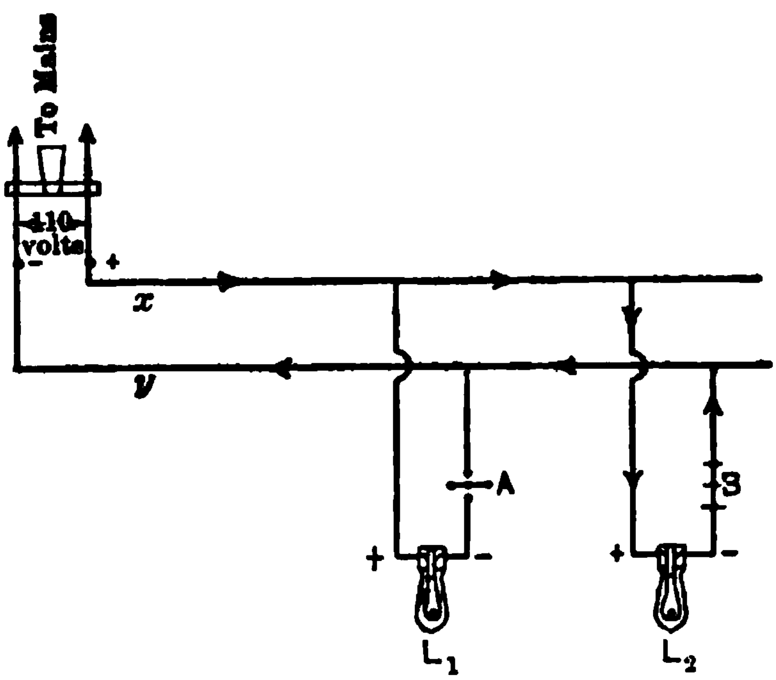


Fig. 1. Current Always Flows from (+) to (−)

Pressure. Volts. When a current of water flows from one point to another in a pipe-system, it is always because there is a hydraulic pressure present causing it to flow. This pressure is usually measured in pounds per square inch. Similarly, when a current of electricity flows from one point to another in an electric circuit, it is because there is an electric pressure present which causes it to flow. This electric pressure is measured in VOLTS. An electric pressure of 1 VOLT is analogous to a hydraulic pressure of 1 lb per sq in. The pressure which causes the ½-ampere current to flow through an incandescent lamp is usually 110 volts. The electric company instals at least two wires in a residence and then maintains an electric pressure of 110 volts between them just as the water company maintains a pressure in the water-pipes. This electric pressure is at all times tending to force electricity from one wire to the other wire across the space between the two wires, just as the water-pressure tends to force the water out from the pipe. The rubber insulation is put on to prevent this flow, very much as the strength and compactness of the iron prevents the flow of water through the walls of the pipe. But when one terminal of a lamp is connected to one wire and the other terminal to the other wire, the electric pressure tending to send a current from one wire to the other, sends a current through the lamp and causes it to glow. We mark the wire bringing the

current to the lamp (+). The wire taking the current away, we mark (−). Thus in Fig. 1, if the current comes in on the wire marked (x), this wire is (+) and the wire (y) is (−). A pressure of 110 volts is maintained which tends to cause a current to flow across from the wire (x) to the wire (y). No current can flow, however, unless some path is afforded between the two wires. For instance, no current is flowing through lamp L_1, because the open switch A makes a gap across which the current cannot pass. Switch B, however, is closed, thus allowing the pressure to force a current from the wire (x) through the lamp L_2 to the wire (y) and back into the street-mains. Of course the electric company maintains the 110-volt pressure between the wires (x) and (y) whether any current is drawn from the wires or not, just as a water company maintains the pressure in the water-mains whether any water is drawn from the pipes or not.

Resistance. Ohms. The fact that a current of only ½ ampere flows through an incandescent lamp when a pressure of 110 volts is applied to it, is due to the RESISTANCE of the fine filament. This resistance of the filament is analogous to the resistance which a pipe of small bore offers to the flow of water. The resistance of an electrical appliance is merely the ratio of the pressure to the current which that pressure can force through it. As an equation, it is expressed

$$\text{Resistance} = \frac{\text{pressure}}{\text{current}}$$

When the pressure is measured in volts and the current in amperes, the resistance is then in ohms. Thus

$$\text{Ohms} = \frac{\text{volts}}{\text{amperes}}$$

Thus, since a pressure of 110 volts forces ½ ampere through an ordinary incandescent lamp, the resistance of the lamp is 110/½ = 220 ohms.

Ohm's Law. This relation between pressure, current and resistance is called OHM'S LAW. It is written in symbols in the three forms

$$R = E/I$$
$$E = IR$$
$$I = E/R$$

where

R = resistance in ohms;
E = pressure in volts;
I = current in amperes.

Example. An electric flat-iron has a resistance of 35 ohms. What current will flow through it when it is put across a 110-volt circuit?

$$I = E/R = 110/35 = 3.14 \text{ amperes}$$

Example. An electric toaster takes 1½ amperes when on a 115-volt circuit. What resistance does it have?

$$R = E/I = 115/1.5 = 76.6 \text{ ohms}$$

Insulators and Conductors. In order that practically no current may leak from one wire to the other, the wires are covered with rubber. This rubber covering offers such high resistance to the flow of an electric current that, although two wires may lie very close to one another with only this rubber between them, practically no current leaks through the rubber from one wire to the other. Materials such as rubber, glass, porcelain, dry wood, etc., have this resisting property and are said to be INSULATORS. Metals, on the other hand, offer very little resistance to the flow of an electric current and are called con-

DUCTORS. A copper wire $\frac{1}{10}$ in in diameter has a resistance of only $\frac{1}{1000}$ of an ohm per foot. Accordingly, because of their low resistance, copper wires are generally used to carry electric currents, and because of its high resistance, rubber is generally used as a covering of the copper wires to prevent leakage from one wire to another. Wire, approved by the National Board of Fire Underwriters and installed according to their rules, will have the proper insulating covering for each installation.

Power. Watts. The flow of an electric current has been likened to the flow of water through a pipe. A current of water is measured by the number of gallons, or pounds, flowing per minute; a current of electricity is measured by the number of amperes. The power required to keep a current of water flowing is the product of the current in POUNDS PER MINUTE by the head, or pressure, in FEET. This gives the power in FOOT-POUNDS PER MINUTE. To reduce to horse-power, it is necessary merely to divide by 33 000. Thus

$$\frac{\text{(pounds per minute)} \times \text{(feet)}}{33\,000} = \text{horse-power}$$

In exactly the same way, the POWER required to keep a current of electricity flowing is the product of the current in AMPERES by the pressure in VOLTS. This gives the power in WATTS.

$$\text{Watts} = \text{amperes} \times \text{volts}$$

The term WATT is merely a unit of power, and denotes the power used when one volt causes one ampere of current to flow. The watts consumed when any given current flows under any pressure can always be found by multiplying the current in amperes by the pressure in volts. Thus, if an incandescent lamp takes 0.5 ampere when burning on a 110-volt line, the power consumed equals

$$0.5 \times 110 = 55 \text{ watts}$$

That is,

$$\text{Power} = \text{current} \times \text{pressure}$$

or

$$\text{Watts} = \text{amperes} \times \text{volts}$$

Example. What power is consumed by a motor which runs on a 220-volt circuit, if it takes 4 amperes?

$$\text{Watts} = \text{amperes} \times \text{volts} = 4 \times 220$$
$$\textbf{Power} = \textbf{880 watts}$$

Incandescent lamps are rated as to the voltage of the line on which they can run, and also as to the amount of electric power it takes to keep them glowing. Thus, a carbon-filament lamp may be rated as a 110-volt, 50-watt lamp. A tungsten-lamp may be rated as a 110-volt, 25-watt lamp. This means that both lamps are intended to run on a 110-volt circuit, but that it takes twice as much power to keep the carbon-filament lamp glowing as it does to keep the tungsten-lamp glowing.

The Power-Equation. The above relation between volts, amperes and watts is usually expressed in the form of an equation:

$$P = IE$$
$$I = P/E$$
$$E = P/I$$

where

P = power in watts;
I = current in amperes;
E = pressure in volts.

Example. What current does a 40-watt tungsten-lamp take when running on a 115-volt circuit?

$$I = P/E = 40/115 = 0.348 \text{ ampere}$$

Power. Kilowatt and Horse-Power. Because the watt is so small a unit of power, being only 0.74 ft-lb per second, a larger unit, the kilowatt, is generally used in connection with machines, etc.

$$1 \text{ kilowatt} = 1\,000 \text{ watts} = 1\tfrac{1}{3} \text{ horse-power}$$

Thus a motor drawing 10 amperes from a 220-volt line would take $10 \times 220 = 2\,200$ watts $= 2\,200/1\,000 = 2.2$ kilowatts.

At 80% efficiency this motor would give out 80% of $2.2 = 1.76$ kilowatts $= 1.76 \times 1\tfrac{1}{3} = 2\tfrac{1}{3}$ horse-power.

Horse-Power-Hour. Kilowatt-Hour. When a man buys mechanical power to run machinery, he has to pay not only according to the horse-power he uses but also according to the number of hours he uses the power. For instance, he may use 40 horse-power for 1 hour and pay $1.20 for it, that is, at the rate of 3 cts for each horse-power-hour. If he uses 40 horse-power for 2 hours he would have to pay twice as much, because he has used the same power twice as long. Another way of stating the same fact is to say that he used twice as many horse-power-hours. For in the first instance he used 40×1, or 40 horse-power-hours, and in the second 40×2, or 80 horse-power-hours. In other words, he did twice as much work in the second case as he did in the first, or received twice as much energy. The unit of work or energy, then, is the HORSE-POWER-HOUR, and is the work done in 1 hour by a 1-horse-power machine.

Example. How much work is done by a machine delivering 15 h.p. when it is run for 8 hours?

1 h.p. in 1 hr does 1 h.p.-hr
15 h.p. in 1 hr does 15 h.p.-hr
15 h.p. in 8 hr does 8×15, or 120 h.p.-hr

That is

$$\text{Work} = \text{horse-power} \times \text{hours}$$

or

$$15 \times 8 = 120 \text{ h.p.-hr}$$

Similarly, electric power is sold by the KILOWATT-HOUR. This unit is the work or energy delivered in one hour by a 1-kilowatt machine.

For lighting purposes electrical energy is usually sold for from 10 to 15 cts per kilowatt-hour. Thus at 12 cts per kw-hr the monthly bill for burning a 40-watt lamp on an average of 5 hours per day would be computed as follows:

For 1 month of 30 days the lamp is burning

$$30 \times 5 = 150 \text{ hours}$$

To use a 40-watt lamp 150 hours consumes

$$40 \times 150 = 6\,000 \text{ watt-hours} = 6\,000/1\,000 = 6 \text{ kilowatt-hours}$$

At 12 cts per kw-hr, 6 kw-hr cost

$$6 \times 12 = \$0.72$$

An instrument called a KILOWATT-HOUR METER is placed in each house to measure the number of kilowatt-hours which each customer consumes. See *M* in Fig. 13 for location of Kilowatt-hour meter, and Fig. 18 for method of connection in typical installation.

Heating-Effect of Current. An electric current always heats the material through which it passes. Examples of this are the incandescent lamp, in which

ie current heats the fine tungsten wire until it glows; the electric heaters for ıafing-dishes, toasters, etc. Even the wires carrying the current to and from ıe lamps are heated by the passage of the current through them. But since the ıating effect for a given current is directly proportional to the resistance of ıe conductor, and the conductors always have very little resistance, the heat-g here is very slight indeed. If conductors of smaller size, and therefore of higher resistance, were used, the heating would be very pronounced; in fact, would soften the rubber insulation and might even produce a temperature high ough to set fire to the building. For this reason The National Board of re Underwriters issues a table specifying the size of wire which must be used r each amount of current. If smaller wire is used, the resistance of it might great enough to raise the temperature to a dangerous degree. On the other nd, if a greater current than allowed by this table is sent over the wire, the nperature will also rise, because the heating of a current is also directly pro-rtional to the SQUARE OF THE CURRENT. Thus, doubling the current which a tain wire is carrying will quadruple the amount of heat which the wire must liate. For this Tables III and IV, see pages 1473 and 1474.

Fuses and Circuit-Breakers. Use is made of the heating effect of a current protect a circuit against too much current, very much as a boiler is pro-ted by a safety-valve against too much pressure. A small piece of fusible tal, generally a mixture of lead and bismuth, is inserted in the circuit in such ay that all the current which passes through the circuit must also pass through

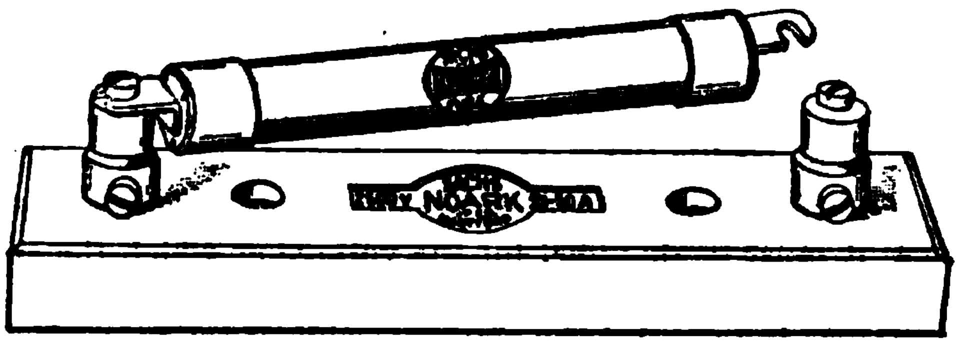


Fig. 2. Enclosed Fuse

piece of metal. This device is called a FUSE. Any current which would be gerous to the circuit melts this fuse, opens the circuit at this point, and thus ects the rest of the circuit from the effects of the current. The cause of large current may be then removed and a new fuse inserted in place of the one. CIRCUIT-BREAKERS are also used to protect a circuit against too much ent. They are AUTOMATIC SWITCHES controlled by an electro-magnet and made in a variety of styles. They operate upon the principle that when electric current passes through a coil of wire it makes a magnet of the coil. coil is so adjusted that when a current of a certain number of amperes es through it, it attracts to itself a small piece of iron. The motion of this e of iron opens the circuit. Fuses and circuit-breakers are thus AUTOMATIC TY-DEVICES required for the protection of all constant-potential systems tever the voltage. Both are for the purpose of protecting the wires from age due to the presence of too much current from any cause whatever. ordinary fuse consists of a porcelain base that has suitable terminals for ting a fuse between the ends of a wire. It must be constructed so that the ing out of a fuse can do no damage, that is, set anything on fire, and placed e it can easily be reached to replace the fuse. Formerly a piece of fuse-wire, d a LINK-FUSE, was used in cut-outs, but the underwriters now require en-d fuses (Fig. 2) or fusible plugs which screw into a receptacle. Fuse-

plugs may be used for currents up to 30 amperes; above that enclosed fuses must be used. Fuse-plugs and enclosed fuses are somewhat more expensive than the link-fuse, but are considered safer. A FUSE CUT-OUT or CIRCUIT-BREAKER is required at or near the place where the wires enter a building, and every circuit of twelve 16-c.p. carbon-lights or of sixteen 40-watt tungsten-lights must be protected by a cut-out. Circuit-breakers are more expensive than fusible cut-outs, and are generally used only on SWITCHBOARDS for large installations and where it is desirable to open the circuit instantly on certain loads, which a fuse cannot be depended on to do with any degree of accuracy, owing to both time and surrounding temperature-factors. Circuit-breakers are also used largely on installations where the variation in load is large and frequent and the repeated burning out of fuse would become expensive not only for renewals but also on account of the time required to replace them.

Lamps. Two kinds of lamps are used for electric lighting, INCANDESCENT LAMPS and ARC-LAMPS. The former are used principally for interior illumination, although sometimes used for street-lighting, especially where the streets are thickly shaded by trees. Arc-lamps are especially adapted for street-lighting and for large interiors where they can be kept concealed or above the range of the eye, as in railway-stations, stores, etc. An incandescent lamp as commonly made consists of a glass bulb containing a simple carbon or a tungsten conductor the ends of which are connected to the source of the electric current. When the current flows through the filament it heats it to such a degree that it becomes incandescent; hence the name of the lamp. The lamps with the filament of finely-drawn tungsten represent the latest type and are superior in every way to those having a carbon filament. Tungsten-lamps require about one-third as much power to produce the same candle-power as carbon-lamps, and have a much longer life.

Voltages. In order that the current shall cause the lamp to give its rated CANDLE-POWER, it must be designed for the voltage at which the system is run. If the voltage of the current is much greater than that for which the lamp is designed it will quickly burn out the filament, while if the voltage of the current is below that of the lamp, it will not give its rated candle-power, a voltage 10% lower reducing the candle-power about one-half. The voltage commonly used for tungsten-lamps is from 100 to 130. Tungsten-lamps are also made for voltages of from 20 to 260. Two to four candle-power lamps, for illuminating signs or decorative purposes, are made for from 10 to 13 volts by ½-volt steps, these lamps being commonly used in series, ten lamps on a 100 to 130-volt circuit. Two 5-watt lamps, 50 volts, are also often used in series on a 100-volt circuit.

Candle-Power. Incandescent lamps of from 100 to 130 volts are commonly made 15, 20, 25, 40, 60, 100, 150, 250, 400 and 500 watts. These lamps average 1 candle-power for every 1.1 watts. For the method of computing the number, size and distribution of tungsten-lamps for illuminating a given room see pages 1476 to 1478.

Arc-Lamps. These are of two kinds, OPEN ARC-LAMPS and ENCLOSED ARC-LAMPS, the latter being generally used for interior illumination. The light from the enclosed arc is much softer and steadier than that from the old-style open arc; there are no sparks, and the life of the carbon is from twelve to fifteen times as great as in the open arc.

"**Direct-Current Open Arcs** usually require about 10 amperes at 45 volts, or 450 watts. The range of voltage is from 42 to 52 for ordinary constant-current arcs. The most satisfactory light is given by from 45 to 47 volts.

rc-lights used for stereopticon-lanterns may use as high as 25 amperes and rovision should always be made in the wiring-plans for such a light for suffi-ently large wires to be installed to carry one and one-half times this current.

"**Direct-Current Enclosed Arcs** consume about 5 amperes at 80 volts, or o watts." Arc-lamps generally require a resistance in series with the arc order to regulate properly. This resistance is usually placed within the struc-re of the lamp, and may be so adjusted that a single lamp can be made to ırn well on any circuit from 100 to 130 volts.

Dynamo-Electric Machines. There are three classes of dynamo-electric ıchines:

(1) GENERATORS for generating an electric current.
(2) MOTORS for converting electrical into mechanical energy.
(3) TRANSFORMERS and ROTARY CONVERTERS.
  (*a*) Transformers for converting one voltage into a higher or lower voltage. Converters and transformers belong to the same class.
  (*b*) Rotary converters for changing alternating currents to direct currents or vice versa.

A DYNAMO is either a motor or a generator. A MOTOR is the same machine as ;enerator, but with the nature of its operation reversed. GENERATORS are two general classes, namely, continuous-current and alternating-current .chines; the latter are commonly called ALTERNATORS. Generators and tors of all kinds vary in voltage, current and speed, according to the pur-e for which they are designed. A TRANSFORMER consists essentially of two ls of wire, one coarse and one fine, wound upon an iron core. Its function o convert electrical energy from one voltage to another. If it reduces the tage it is known as a STEP-DOWN transformer, and if it raises it, it is known ı STEP-UP transformer. A transformer has no moving parts and requires no endant.

Kinds of Currents Produced. There are two kinds of electrical currents ımonly used for light and power in buildings, (1) DIRECT CURRENTS, and (2) ERNATING CURRENTS.

A direct current is uniform in strength and direction, while an alternating rent rapidly rises from zero to a maximum, falls to zero, reverses its direction, ıins a maximum in the new direction and again returns to zero. A complete of these changes is called a CYCLE. The number of times the current goes ough these changes during each second is called the FREQUENCY of the current. frequency commonly used for incandescent lighting is 60 cycles per ond; that is, the current goes through the above changes in value 60 times second. A frequency of 25 cycles is also in common use, especially for run-g motors, although it is not so satisfactory for use with incandescent lights. direct current is likened to the steady flow of water through a pipe-system, alternating current may be likened to the rapid surging back and forth of er in a pipe-system. More difficulty was experienced in utilizing these rapid ;es of electricity than in developing direct-current apparatus. Consequently use of the alternating current was retarded but is now becoming general. advantages of alternating over direct currents are: (1) Greater simplicity ynamos and motors, no commutators being required in some types; (2) the ibility of obtaining high voltages by means of transformers for cheapening cost of transmission; (3) the facility of transforming from one voltage to ther, either higher or lower, for different purposes."*

* Adapted from Kent's Pocket Book.

Table I. Average Current Taken by Direct-Current Motors

Horse-power	Amperes on 110-volt line	Amperes on 220-volt line	Horse-power	Amperes on 110-volt line	Amperes on 220-volt line
¼	3	1.5	25	186	93
½	5.4	2.7	30	222	111
1	9	4.5	35	260	130
2	17	8.5	40	296	148
3	25	12.5	50	....	185
5	40	20	60	....	220
7½	58	29	75	....	275
10	76	38	85	....	312
15	114	57	100	....	366
20	150	75	....	....	....

The current taken by single-phase alternating-current motors can be found by noting the current taken by a direct-current motor of the same size and voltage, and dividing this current by the power-factor of the alternating-current motor. To find the current taken by each terminal of a three-wire, three-phase alternating-current motor, divide the current taken by a single-phase alternating-current motor of the same size and voltage by 1.73.

Example. What current is taken by a 5-horse-power, alternating-current, 220-volt, induction-motor of 80% power-factor?

Solution. A 5-horse-power, direct-current, 220-volt motor takes 20 amperes. A single-phase, 5-horse-power, 220-volt motor of 80% power-factor takes 20/.80 = 25 amperes.

Electric-Lighting Systems Commonly Used for Supplying the Electrical Energy to Lamps

Direct-Current, Constant-Potential Systems. The systems most used in America are:

(1) Two-wire system largely used for incandescent lighting from small plants, as for a large office-building or factory. It is usually operated at 110 volts.

(2) Three-wire system used in small towns for the lighting of buildings from the public mains, usually operated at 220 volts. Also in large cities with underground conduit-system. See pages 1466 to 1468.

Five-wire and seven-wire systems with high voltage have been used in Europe, but very little in America.

Alternating-Current, Constant-Potential Systems. There are two systems:

(1) Single-phase system. Current transmitted to building at from 1 000 to 2 000 volts and reduced to from 50 to 110 volts by a transformer. The term phase is used in connection with alternating-current systems only in the sense of circuit. Thus a single-phase system means an alternating-current system sending out power from one circuit only of the generator. A three-phase system has three circuits.

(2) Three-phase system. Three or four wires are used. This system is most used for lighting from public plants, principally because it enables both lights and motors to be operated from the public dynamo and is the most economical in wire. (See Table II.) Both of these systems are used for incandescent lighting and for power from central stations. For a comparison of a three-wire direct current with a three-phase, three-wire alternating current, see pages

168-9. An alternating current may be changed to a direct current at a sub-ation by a rotary converter or by a mercury-arc rectifier. The latter is very nerally used in garages in order to convert an alternating current into a direct rrent for charging storage-tteries.

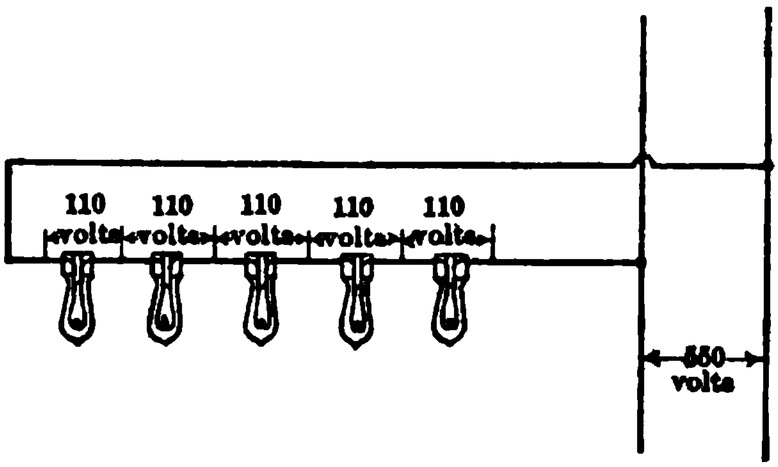
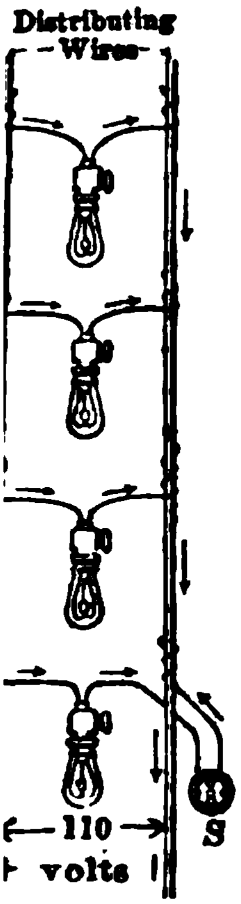


Fig. 3. Five Lamps in Series on a 550-Volt Line. Each Lamp has a Voltage of 110 Volts Across It

Methods of Connecting imps. There are three ways connecting lamps to the dis-bution-wires: (1) in series; in parallel; and (3) in rallel series.

(1) **Lamps in Series.** mps are said to be connected series when they are arranged after the other, so that same current flows through the lamps. The most com-n example of this system is the lighting of electric cars and the stations on electric-railway line. The voltage of such lines is usually 550 volts. Since ordinary incandescent lamp requires but 110 volts, five of these are placed eries as in Fig. 3. Each lamp now has a pressure of 110 volts across it, and the set of five lamps requires 550 volts across it, and so can be placed across the railway supply-wires. When lamps are arranged in series the total resistance of the circuit is the sum of the resistances of the several parts, and the voltage required to force the current through a number of lamps in series is the sum of the voltages required for the separate lamps. Thus the voltage required to supply the proper current for four 52-volt lamps is $4 \times 52 = 208$ volts. Arc-lamps for street-lighting are often connected in series, but incandescent lamps are very seldom connected in series except as described above or for decorative purposes or electric signs. Where lamps of low voltage, as in signs, etc., are used on 110-volt systems it is necessary to connect them in series. The underwriters do not approve connecting incandescent lamps in series. The series system requires that the same current flow through each lamp, and if one lamp burns out the circuit is broken and all of the lamps will go out, unless some provision is made for maintaining the circuit around the dead lamps.



L. Four Lamps Parallel. Each np Has the Full-Pressure of 110 ts Across It

(2) **Lamps in Parallel.** This is the common method of connecting incandescent lamps. It is illustrated in Fig. 4. With this system the pressure in each lamp is the same as in the distributing lines, and any lamp may be turned on or off without affecting the other lamps. For this system the PRESSURE or voltage must be kept constant, while the current or quantity of electricity flow-n the lines will depend upon the number of lamps that are burning. with twelve 16-candle-power lamps of 110 voltage on a parallel circuit, lamp requiring 0.51 ampere when all the lamps are burning, a current of amperes, or 673.2* watts, will be required. With but one lamp burning,

* Watts being equal to amperes times voltage.

a current of only 0.51 ampere will flow. The voltage, however, must be the same for one lamp as for the twelve. For lamps in parallel, therefore, a CONSTANT-POTENTIAL system is required. The current for lamps in parallel may be turned on or off at the lamp, or a switch-loop may be run any distance and the contact made by a switch (*S*) as for the lower lamp (Fig. 4).

(3) **Lamps in Parallel Series.** This method is a combination of the other two. Parallel lines are run as in the parallel system, but two or more lamps

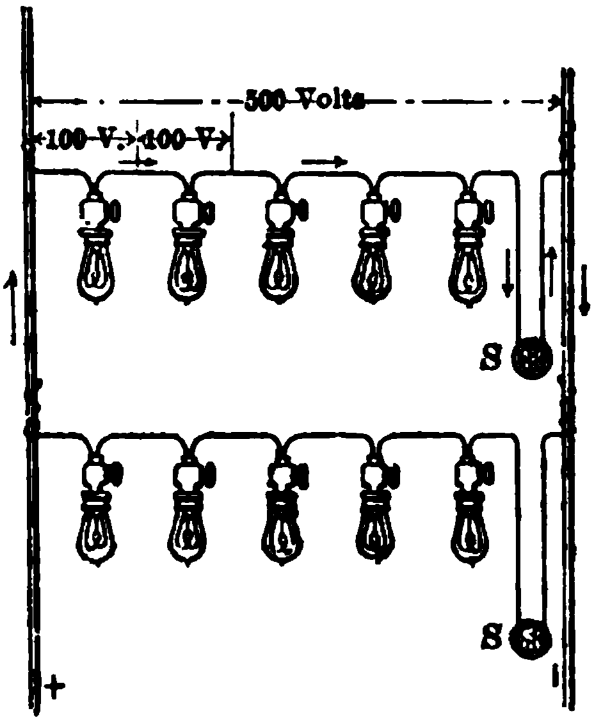


Fig. 5. Lamps in Parallel Series

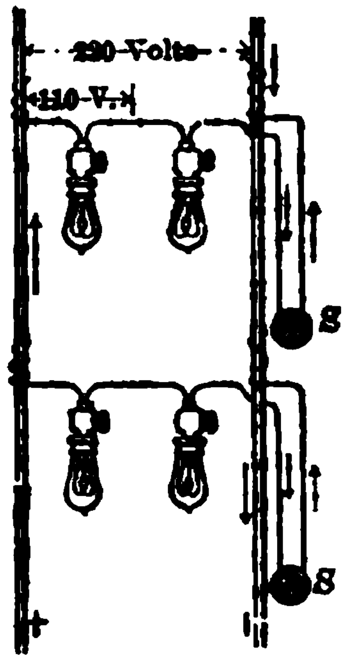


Fig. 6. Lamps in Parallel Series

are connected in series between them as in Figs. 5 and 6. This method of connecting lamps is used principally in places where it is desired to operate lamps on a power system. Fig. 5 shows a series of five lamps operated on a 500-volt system and Fig. 6 a series of two lamps on a 220-volt system using 110-volt lamps. Any number of series may be connected across the mains, each series being independent of the others. But in each series if one light burns out, the others in the same series will be useless, and one lamp alone cannot be used. The sum of the voltages of the lamps in series must be approximately equal to the voltage between the mains. There are a number of special cases in which this method of connection may be used.

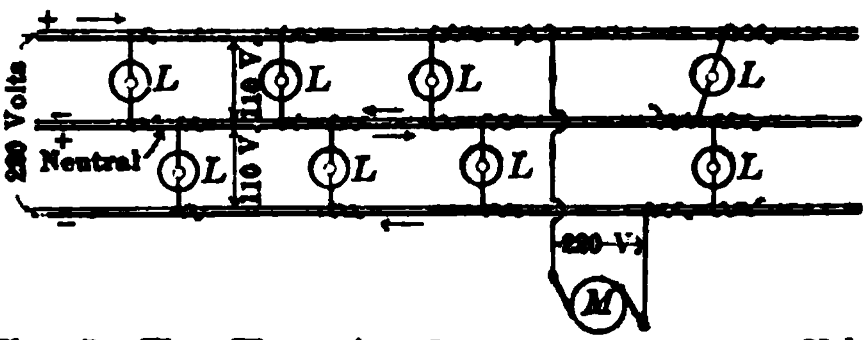


Fig. 7. The Three-wire Edison System. 220 Volts Between Outside Wires; Only 110 Volts Between Either Outside Wire and Neutral Wire

The Edison Three-Wire System. Figs. 4, 5 and 6 are examples of the two-wire system of distribution, which is the system recommended for average-sized office-buildings, apartment-houses, theaters and stores. Where power for motors is to be taken from the same plant as the lighting current, and where the power is not too great a portion of the capacity of the installation, this two-wire system may also be used. Separate mains, however, should under all circumstances be run for the motors, as the variation in load and, consequently, the current-demand on the mains would cause a very appreciable fluctuation in

andle-power of the lamps, if on the same mains with the motors. Where com-paratively long lines are required and the amount of current to be supplied is

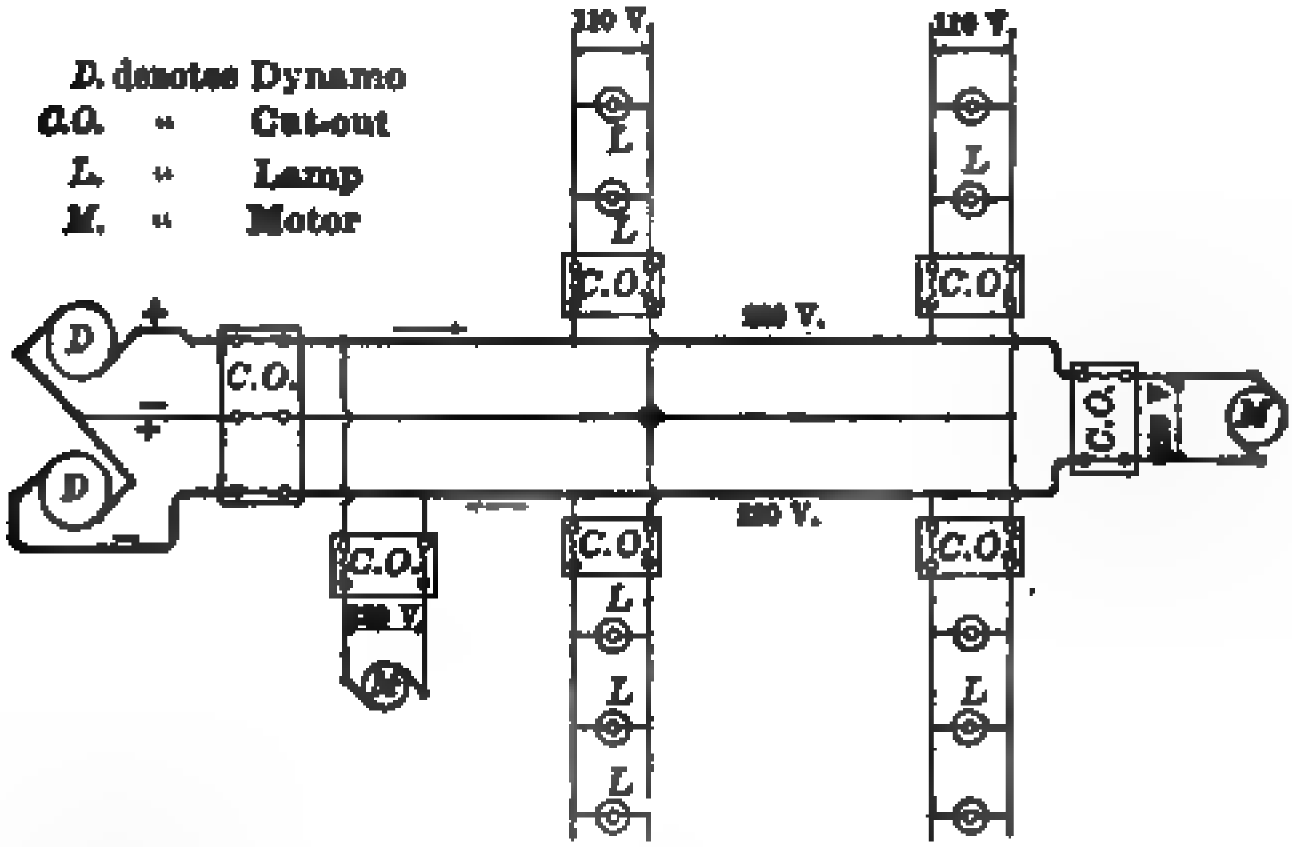


Fig. 8. Example of Three-wire System of Wiring

ge the THREE-WIRE SYSTEM is used. By this system two voltages or pressures be supplied, 110 and 220 volts being those generally adopted, the 110-volt uit supplying the arc and andescent lights and the -volt circuit the motors. . 7 shows how the wires run and connections le. The pressure between two outside wires is the voltage transmitted from generator, usually 220 s for interior wiring. current in these two s flows in opposite direc-s. The middle wire, d the NEUTRAL WIRE, a one side of two cir-, the current from one it tending to flow in direction and that from ther circuit in the oppo-direction; consequently currents of the same gth, in amperes, are ig in both circuits they alize each other in the e wire and there will current flowing in this

Fig. 9. The Wiring of a Cabinet. Showing How to Divide a Three-wire System into Six Two-wire Circuits, Three Circuits to Each Leg

With a current of 10 amperes flowing in one circuit and one of 6 es in the other circuit, the current flowing in the neutral wire will be 4

amperes. To obtain the greatest benefit from this system, it should always be installed so that there will be nearly the same load or number of lamps on each side of the neutral wire. Even then there will be times when more lamps will be burning on one side than on the other, so that it is necessary to give some size to the neutral wire. The neutral wire is seldom made less than one-half the cross-section of the outer wires. For distributing mains in buildings carrying lamps only, the neutral wire should be of the SAME SIZE as the outer wires. From Table II it will be seen that the three-wire system effects a considerable saving in copper, amounting to fully 60% of the ordinary two-wire 110-volt system. As a rule, in supplying current for light and power from one plant, the main wires only are arranged on the three-wire system and the distributing wires are run on the two-wire system as in Fig. 8. When using the three-wire system for lighting only, the three wires are usually run no farther within the building than to the centers of distribution, and from these centers two wires are run for each circuit, the circuits being divided as equally as possible on the two sides of the three-wire system as shown by Fig. 9. Three-wire mains are now very commonly used where the current exceeds 100 amperes. When motors are operated from the three-wire system they are usually connected only to the outside wires. Motors used on three-wire incandescent-lighting systems should be wound for 220 volts.

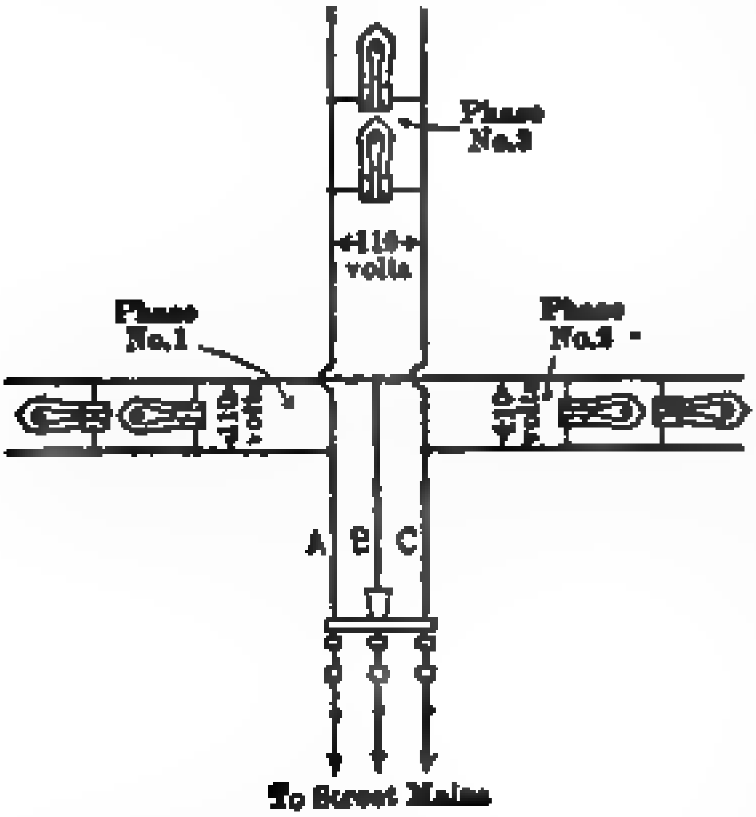


Fig. 10. Three-phase, Three-wire System, Alternating Current. Compare with Fig. 11

Fig. 11. Three-wire System, Direct Current. Compare with Fig. 10

Comparison of the Three-Phase and Three-Wire Edison Systems. The wiring for the Edison three-wire direct-current system is the same as that for the three-wire, three-phase alternating-current system, the only difference being that the voltage BETWEEN ANY TWO WIRES of a three-phase system is the same Thus in Fig. 10 which represents a three-wire, three-phase system the voltage between the wires *A* and *B* (phase No. 1)

110 volts; between B and C (phase No. 2) is 110 volts; and between A d C (phase No. 3) is 110 volts. But in Fig. 11, which represents a threere direct-current system, in which the voltage across A and B, and B and is 110 volts, the voltage across A and C is 220 volts or twice that across her leg.

Table II. Relative Weight of Copper Required in Different Systems for Equal Effective Voltage

Direct-current, ordinary two-wire system	1.000
Direct-current, three-wire system, all wires of same size	0.375
Direct-current, three-wire system, neutral, one-half size	0.313
Alternating-current, single-phase two-wire system	1.000
Three-phase three-wire	0.750
Three-phase four-wire	0.333

Wire-Calculations

Vire-Gauges.* As the diameter of wires is ordinarily designated by the nber of a wire-gauge, and as there are a number of wire-gauges in common use, ie knowledge of those used for copper wire is necessary. The Brown & Sharpe, 3. & S., gauge (see page 1474) is almost exclusively used in America in connec- with electrical work, except where the size of the wire is designated in cirır mils. The sizes of wire given by this gauge range from No. 0000 (0.46 in) Vo. 40 (0.0031 in), but No. 14 is the smallest size permitted for interior wiring. No. 10 wire has a diameter of about 1/10 in and its resistance per 1 000 ft ery nearly 1 ohm. For any given number of this gauge a wire three numbers ier has very nearly half the cross-section, and one three numbers lower has e the cross-section; thus a No. 13 wire has very nearly one-half the crossion of a No. 10 wire, and a No. 7 has twice the cross-section of a No. 10, our times that of a No. 13.

he Circular-Mil Wire-Gauge. This gauge was designed by the engineerdepartment of the Edison Company especially for the designation of copper for electrical work, and is now in general use in this country. In practice B. & S. gauge is commonly used for designating wires up to No. 0 or No. 00, all wires above that size are designated by circular mils (c.m.). The size ire required is often determined in circular mils and designated by the correding B. & S. gauge-number, which is readily done by means of Table III, 1473. Copper wire is sold by the pound if bare or of the numerous her-proof varieties, but rubber-covered wire is sold by the 1 000 ft.

ie basis of the circular-mil gauge is the area of a wire 1/1000 in in diameter il = 0.001 in); consequently, 1 c.m. = 0.0000007854 sq in. As the areas of es vary as the squares of their diameters, it follows that the sectional area vire 2 mils in diameter = 4 c.m., of a wire 10 mils in diameter 100 c.m., and 1.

hen wires are designated by circular mils, the SECTIONAL AREA and not the eter is generally given, c.m. always referring to sectional area. The diamof a wire in MILS OR IN THOUSANDTHS OF AN INCH = square root of its area RCULAR MILS.

us the diameter of a wire of 3 600 c.m. = 60 mils, or 0.060 in.

e diameter of a wire 14 400 c.m. = 120 mils = 0.12 in.

e area of a wire 0.162 in in diameter, or 162 mils, $= 162^2 = 26\,244$ c.m.

* For other gauges, see pages 401, 402, 403, 1473, 1509, 1510, 1512 and 1600.

To reduce circular mils to square inches. Multiply by 7 854 and point off ten places of decimals. Thus, 5 000 c.m. = 7 854 × 5 000 = 0.0039270000 sq in.

To obtain the sectional area of a square or rectangular bar in circular mils. Multiply together its dimensions in mils and the product by 1.273.

Example. What is the sectional area in circular mils of a bar ⅛ in × ¼ in?

Solution. ⅛ in = 0.125 in = 125 mils, ¼ in = 0.250 in = 250 mils; 125 × 250 × 1.273 = 39781.25 c.m.

The weight of bare copper wire per 1 000 ft = c.m. × 0.003027 lb. Thus the weight of 1 000 ft of copper wire having a sectional area of 2 000 c.m. = 0.003027 × 2 000 = 6.054 lb. Table IV, page 1474, gives the dimensions and weights of bare copper wire from No. 18 to No. 0000 B. & S.

Carrying Capacity of Copper Wire. The safe carrying capacity of copper wire for interior wiring is practically fixed by the underwriters, and if the capacity-limits given in the table published by them are exceeded it would tend to destroy the right to recover insurance in case of fire. The safe carrying capacity of rubber-covered and weather-proof wires given by the National Board of Fire Underwriters is shown by Table III, page 1473. The lower ampere-capacity assigned to rubber-covered wires is due to the fact that the rubber insulation would deteriorate in quality under a temperature as high as that allowed for weather-proof wire; that is, the rubber covering makes necessary a lower rate of heat-development than is required for safety from fire. No wire smaller than No. 14 may be used under insurance-rules, except that No. 16 may be used for flexible cord and No. 18 for fixture-wiring. Nos. 13, 11, 9 and 7 are not usually carried in stock and can only be purchased on special order. Rubber-covered wire must be used for service-wires, for molding-work and in damp places; it is more expensive than weather-proof wire. The latter wire may be used in open or exposed places and for outside line-wires.

Drop of Potential. When an electric current flows through a wire of any appreciable length the pressure becomes reduced by the resistance of the wire, so that if the current enters the wire at, say, 110 volts, at the extreme end of the circuit it will be somewhat less, depending upon the length and sectional area of the wire. This loss in voltage is called DROP OF POTENTIAL. Drop of potential corresponds to LOSS OF HEAD in hydraulics. As a drop of voltage materially below that for which the lamps are designed means diminished candle-power, it is very important that the wires be proportioned so that the drop shall not be sufficient to affect the illumination. The table for safe carrying capacity for wires has nothing to do with the drop of potential which these currents will cause in the wires. Accordingly, mains and distributing wires may be capable of carrying the number of amperes in accordance with Table III, page 1473, and yet cause a drop of potential of such magnitude that the most distant lamps will burn only at a dull red. It is therefore necessary, in computing the size of these mains and distributing wires, to consider two things:

(1) That the wire is large enough, according to the underwriters' table, to carry the current safely.

(2) That the potential drop from the generator to the farthest lamp shall not be excessive. An excessive drop in voltage also means increased cost for light and not enough copper in the wires.

Where the current is supplied from the public mains it is usual to specify a 2% drop, but where the current is produced cheaply, as by a dynamo on the premises, a 3% or 5% drop may be allowed. Not more than a 5% drop on short distances should be permitted, even where very cheap work is desired. The

drop in volts (not in percentage) = current in line × resistance of line, or drop
n volts = amperes × ohms.

Example. What will be the drop in a circuit of No. 14 copper wire 280 ft
ong, supplying nine lamps, requiring 4.5 amperes?

Solution. From Table IV, page 1474, it is found that the resistance of No.
4 wire is 2.527 ohms per 1 000 ft; hence for 280 ft it will be 2.527 × 0.280 =
.7075 ohm, and drop in volts = 4.5 × 0.7075 = 3.1837 volts. The voltage for
his current (0.5 ampere per lamp) will be about 110; consequently the per-
entage of drop = 3.1837/110 = 2 9/10%, nearly. A 2% drop on a pressure of
10 volts is 2.2 volts.

Load-Center. The meaning of this term may best be illustrated by an exam-
le. Let Fig. 12 represent a circuit carrying six lamps, the first lamp being

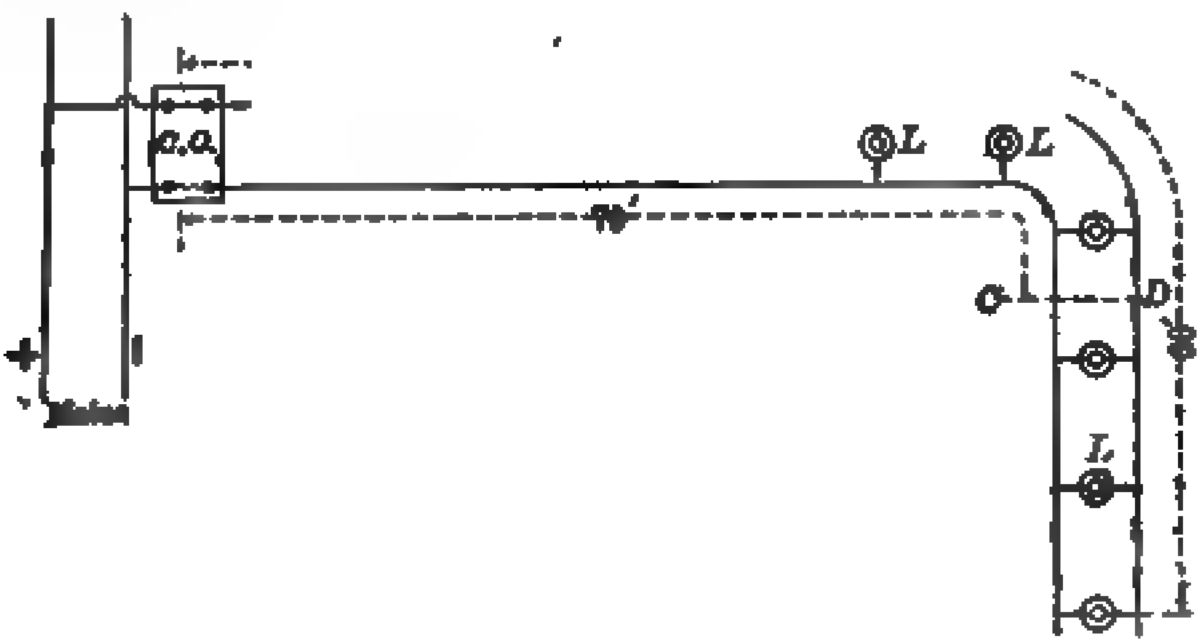


Fig. 12. The Point *D* is the Load-center

ft from the cut-out, or source of supply. The whole of the current must be
nsmitted through this 40 ft, but from that point it will gradually fall off, and
average current will only extend to the point *CD*, halfway between the
reme lamps. Or, in other words, the load-center is analogous to the center
gravity of the lamps on the circuit. The load-center determines the length of
line in the rules for finding the necessary size of wire.

Distributing Centers are the points in a building where the cut-out cabinets
located and the branch circuits taken off.

Calculations for Size of Wire for Incandescent Lighting. The sizes of
s for interior lighting are or should be always determined on a basis of a fixed
p of potential, usually 2 volts on the distributing circuit and from 2 to 3
s on the feeders or mains.* The size of wire may be determined either in
s of its sectional area in circular mils or in terms of its resistance in ohms
1 000 ft. Knowing the sectional area in circular mils, one may find the
esponding gauge-number from Table III, page 1473, or if the resistance in
s per 1 000 ft is known, the corresponding gauge-number may be found
Table IV, page 1474.

* Many municipal lighting companies require that there shall be no more than 2%
drop in the wiring for interior lighting.

The formula for circular mils is as follows:

$$\text{Circular mils} = \frac{10.4 \times 2d \times N \times c}{v} \tag{1}$$

The formula for resistance per 1 000 ft of wire is

$$\text{Resistance} = \frac{1\,000\,v}{N \times c \times 2d} \tag{2}$$

In both these formulas d = distance in feet, one way, from cut-out to load-center (see page 1471) for distributing wires, or from entrance cut-out or source of current to distributing center for main lines or feeders. c = current in amperes PER LAMP. N = number of lamps supplied. v = drop in volts. Both formulas apply to any voltage and to any two-wire system. To use these formulas for the ordinary three-wire system, let N = maximum number of lamps on ONE SIDE of the neutral wire and DOUBLE THE DROP IN VOLTS. The neutral or middle wire should be of the same size as the outside wires.

Example. The distance from the cut-out to load-center of a circuit carrying sixteen 40-watt, 110-volt lamps is 50 ft. What size of wire should be used for a drop of 2 volts?

Solution. $d = 50$; $N = 16$; $c = 40/110 = 0.364$; and $v = 2$.
By Formula (1),

$$\text{Circular mils} = \frac{10.4 \times 100 \times 16 \times 0.364}{2} = 3\,030$$

Table III, page 1473, shows that the next larger size of wire is 4 107 c.m., equivalent to a No. 14 wire.
By Formula (2),

$$\text{Resistance per 1 000 ft} = \frac{1\,000 \times 2}{12 \times 0.364 \times 100} = 4.59$$

which we see from Table IV, page 1474, is about the resistance of a No. 16 wire; but as No. 14 is the smallest wire permitted that size must be used.

Example. The distance from the entrance cut-out, where the wires enter the building, to the main distributing center of a building is 100 ft. The total number of 16-candle-power, 110-volt carbon-lamps supplied is ninety. What is the size of the mains that should be used on the two-wire system with a drop of 2 volts? (A 16-candle-power 110-volt carbon lamp takes approximately 0.51 ampere.)

Solution. $d = 100$; $N = 90$; $c = 0.51$; $v = 2$
By Formula (1),

$$\text{Circular mils} = \frac{10.4 \times 200 \times 90 \times 0.51}{2} = 47\,800$$

In Table III it is seen that No. 3 wire must be used. If a drop of 3 volts is allowed the sectional area required will be 33 048 c.m., which requires a No. 5 wire. The weight per 1 000 ft of No. 3 weather-proof wire (Table IV) is 200 lb and of No. 5 wire 125 lb; consequently, the SAVING IN WEIGHT OF WIRE by using a drop of 3 volts instead of 2 is 75 lb, or 37½% of 200, and as wire is sold by the pound, the SAVING IN COST with a 3% drop ranges from 30 to 40% of a 2% drop.

Example. With the same conditions as given in the preceding example what is the size of the wire that will be required for the ordinary three-wire system with 2% drop?

Table III. Carrying Capacity of Wires and Cables

FOR INTERIOR CONDUCTORS, ALL VOLTAGES

From the National Electrical Code

No. of wire, B. & S. gauge *	Circular mils	Capacity in amperes	
		Rubber-covered	Weather-proof
18	1 624	3	5
16	2 583	6	10
14	4 107	15	20
12	6 530	20	25
10	10 380	25	30
8	16 510	35	50
6	26 250	50	70
5	33 100	55	80
4	41 740	70	90
3	52 630	80	100
2	66 370	90	125
1	83 690	100	150
0	105 500	125	200
00	133 100	150	225
000	167 800	175	275
0000	211 600	225	325
Cables	200 000	200	300
"	300 000	275	400
"	400 000	325	500
"	500 000	400	600
"	600 000	450	680
"	700 000	500	760
"	800 000	550	840
"	900 000	600	920
"	1 000 000	650	1 000
"	1 100 000	690	1 080
"	1 200 000	730	1 150
"	1 300 000	770	1 220
"	1 400 000	810	1 290
"	1 500 000	850	1 360
"	1 600 000	890	1 430
"	1 700 000	930	1 490
"	1 800 000	970	1 550
"	1 900 000	1 010	1 610
"	2 000 000	1 050	1 670

A current of one ampere will supply two 16-candle-power carbon lamps.

olution. In this case we use one-half of N, or 45, and $2\,v$ instead of v; then

$$\text{Circular mils} = \frac{10.4 \times 200 \times 45 \times 0.51}{4} = 11\,920$$

just ONE-FOURTH the section required for the two-wire system. The size vire required is No. 8; a No. 9 would answer if it could be had. Comparing weight of wire required with the two-wire system gives two No. 3 wires ghing 400 lb per 1 000 ft, and with the three-wire system three No. 8 wires ghing 207 lb; hence, the saving in cost is nearly 50% and if No. 9 wire were inable the saving would be 55%. With a drop of 3% (3.3 volts) the cir- r mils required for the three-wire system $= \dfrac{10.4 \times 200 \times 45 \times 0.51}{6.6} = 7\,230$,

* For other gauges, see pages 401, 402, 403, 1469, 1509, 1510, 1512, and 1600.

requiring No. 10 wires. The current in amperes in the two-wire system $= N \times c = 45.9$, and in the three-wire system $\frac{1}{2} N \times c = 22.95$. Referring to Table III it is seen that the smallest size of weather-proof wire permitted for 45.9 amperes is No. 8; consequently, No. 8 wire could be used with the two-wire system and comply with the underwriters' rules, but the drop in potential would be $45.9 \times 0.2 \times 0.6285$ (amperes × resistance of line) $= 5.77$ volts; or over 5%.

For the three-wire system, the current being 23 amperes, the smallest weather-proof wire permitted by Table III is No. 12, which would give a drop of 7.4 volts, or 3.8 volts on each side, or about 3½% of the lamp-voltage. Except on very short lines a 2% drop will always demand larger wires than required by the underwriters, and this is also usually true of a 3% drop.

Table IV. Dimensions, Weights and Resistances of Copper Wire

Gauge-number, B. & S.	Diameter in mils	Area in cir. mils	Area in sq in	Weight in lb per 1 000 ft		Ohms per 1 000 ft at 20° C. or 68° F.
				Bare wire	Weather-proof* wire	
0000	460	211 600	0.166190	640.73	800	0.04893
000	410	167 800	0.131790	508.12	666	0.06170
00	365	133 100	0.104520	402.97	500	0.07780
0	325	105 500	0.082887	319.74	363	0.09811
1	289	83 690	0.065732	253 43	313	0.1237
2	258	66 370	0.052128	200.98	250	0.1560
3	229	52 630	0.041339	159.38	200	0.1967
4	204	41 740	0.032784	126.40	144	0.2480
5	182	33 100	0.025999	100.23	125	0.3128
6	162	26 250	0.020618	79.49	105	0.3944
7	144	20 820	0.016351	63.03	87	0.4973
8	128	16 510	0 012967	49.99	69	0.6271
9	114	13 090	0.010283	39.65	....	0.7908
10	102	10 380	0.008155	31.44	50	0.9972
11	91	8 234	0.006466	24.93	....	1.257
12	81	6 530	0.005129	19.77	31	1.586
13	72	5 178	0.004067	15.68	....	1.999
14	64	4 107	0.003225	12.44	22	2.527
15	57	3 257	0.002558	9.86	....	3 179
16	51	2 583	0.002028	7.82	14	4.009
17	45	2 048	0.001608	6.20	....	5.055
18	40	1 624	0.001275	4.92	11	6.374

* Approximate weight of weather-proof line-wire for outdoor work is 10% less than here given.

To find the smallest size of wire that will comply with the underwriters' rules it is only necessary to compute the total current in amperes, and from Table III select the wire having a capacity equal to or next above the required number of amperes. Table VI shows at a glance the maximum number of 16-candle-power 110-volt carbon lamps permitted by the National Code.

Formulas (1) and (2), page 1472, may also be used for MOTOR-WIRING, if the required current in amperes is known, by substituting the given number of amperes for $N \times c$.

Table V. Maximum Length of Line for Given Number of Lamps that can be Used with a Two-Per-Cent Drop. Two-Wire System

Based on ½ ampere per carbon-lamp. One 32-candle-power carbon-lamp = two 16-candle-power carbon-lamps. Four 40-watt tungsten-lamps = three 16-candle-power carbon-lamps

No. of wire, B. & S. gauge	Number of 16-candle-power, 110-volt carbon-lamps								
	4	6	8	10	11	12	16	20	24
	Maximum length of line, one side, in feet								
14	209	139	104	83	76	70	52	42	35
12	.....	221	166	133	120	110	83	66	55
10	.....	.....	264	211	192	176	132	105	88
8	.....	.....	.....	326	297	272	204	163	136
6	.....	.....	.....	.....	......	440	334	267	220
	Number of 16-candle-power, 110-volt lamps								
	30	36	40	50	60	70	80	90	100
	Maximum length of line, one side, in feet								
12	44	37	.....	.....	......	......	......	......	......
10	70	58	52	42	......	......	......	......	......
8	109	91	81	65	54	37	40	......	......
6	178	148	133	107	89	76	66	59	53
5	225	187	168	135	112	96	84	75	67
4	.....	236	212	170	141	121	106	94	85
3	.....	.....	268	214	180	153	134	119	107
2	.....	.....	.....	270	225	193	169	150	135
1	.....	.....	.....	.....	285	243	213	190	170

For three-wire mains with 220 volts between outer wires and same number of lamps on ch side, length of wire may be increased four times.

Table VI. Maximum Carrying Capacity of Wires in Terms of 16-Candle-Power 110-Volt Lamps, However Short the Wires May Be

Based on ½ ampere per lamp
Four 40-watt tungsten-lamps = three 16-candle-power carbon-lamps

No. of wire, B. & S. gauge	Number of lamps		No. of wire, B. & S. gauge	Number of lamps	
	Rubber-covered	Weather-proof		Rubber-covered	Weather-proof
14	24	32	4	130	184
12	34	46	3	152	220
10	48	64	2	180	262
8	66	92	1	214	312
6	92	130	0	254	370

Example. What should be the size of the wires to be run to a motor that requires 30 amperes at 220 volts and is situated 200 ft from the distributing pole, the drop in volts not to exceed 2%?

Solution. Using Formula (1), and substituting 30 for $N \times c$, we have

$$\text{Circular mils} = \frac{10.4 \times 400 \times 30}{4.4} = 28\,400$$

which requires a No. 5 wire. Either the watts or the current in amperes is stamped on every motor. If watts are given, the current in amperes may be found by dividing the watts by the voltage. If kilowatts are given, multiply by 1 000 and then divide by the voltage.

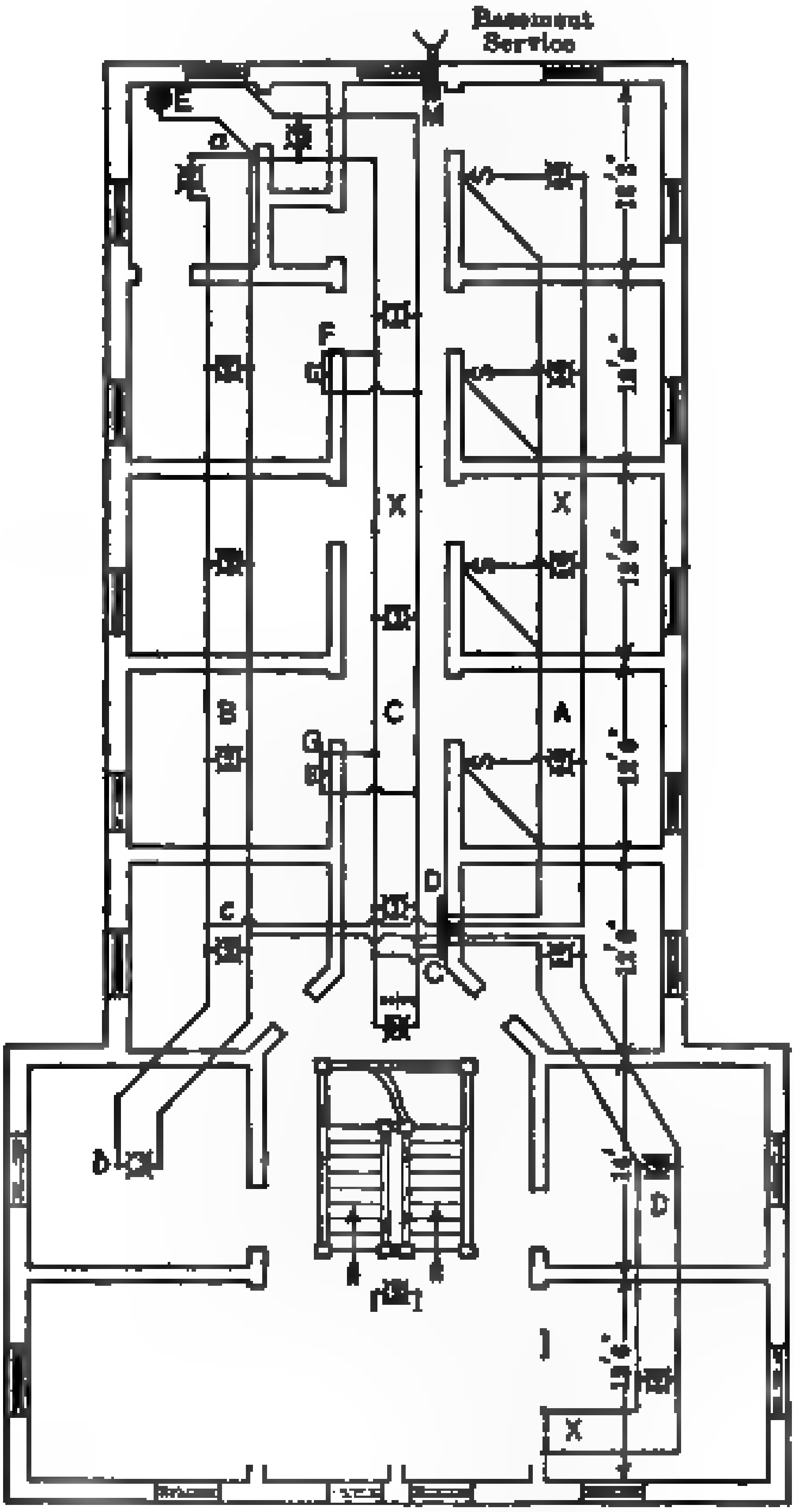


Fig. 1[illegible]. Wiring-diagram for Second Story. For Meaning of Symbols, see pages 1484–5.

Wiring-Tables. Several forms of wiring-tables which are very useful to electricians are published in various books on electricity. For ordinary interior wiring for 110-volt, 16-candle-power carbon-lamps, Table V, computed by Mr. Kidder, will show at a glance the number of wire, B. & S. gauge, required to supply the given number of lamps by first ascertaining the length of line (one way) through which the average current flows, as explained under LOAD-CENTER. (See page 1471 and Fig. 12.)

Simple Example of Wiring. To show the method of wiring an ordinary building for incandescent lighting we will take a two-story building having a floor-plan as shown in Fig. 13. Most of the light-outlets are on the ceiling and are indicated by a small circle. The outlet marked *E* is a special outlet for heating, etc., and must be described in the specifications. Let us assume it is to take 320 watts. This is equivalent to adding eight 40-watt lamps to this circuit. *F* and *G* are wall-outlets. The meanings of the symbols used are explained on pages 1484–5. The numbers

nd 2 inside the circles denote the number of 16-candle-power carbon-lamps .he outlet. The same number of 25-watt or 40-watt tungsten-lamps may ays be used without overloading the circuits. See pages 1398 and 1399 Standard-Wiring Symbols. The current to be obtained from the wires of public lighting company, which carry a current at 220 volts between the side wires, and at 110 volts between either outside wire and the neutral e. The feed wires for the building should enter through the alley-wall at ıut the level of the second ır and should drop in the tition just inside the wall the main fuse-block and tch, which should be in small cabinet and the .er (*M*). The distribu-ı-cabinet should be lo-ed near the center of the lding, say at *DC*, and re should be a cabinet each story. From this inet we will run four cir-s for each story, which indicated by the letters *B*, *C* and *D*. Circuit *A* ws the wires run for a .ch on the wall near the r of each of four rooms ontrol the lights in those ns. All of the lights on uit *C* shou'd be controlled ceys in the lamp-sockets. The lights on circuits *B* and *D* are not switched, ·pt the outlet at head of stairs, which is controlled by a snap or push-button ch at *S*. For a first-class job all of the four circuits would be controlled cnife-switches in the cabinet, as shown in Fig. 14, but this is not absolutely ssary.

F.F. FUSE-PLUGS S.S. KNIFE-SWITCHES

Fig. 14. Cabinet-wiring for Knife-switch Control

ize of Wires. The load-center of circuits *A*, *C*, and *D* would be at about points marked *X* (Fig 13). For circuit *B* take one-half the distance *ab* add to it the distance from *c* to the cabinet In figuring the length ne, 6 ft should be added for the drop from ceiling to the cabinet. Let us as-e that tungsten-lamps are to be used In computing the current taken by ı lamp it is always assumed that no smaller than a 40-watt tungsten is used. drop-lights, marked ○ would probably be 25-watt lamps, but must be ıted as 40-watt, according to the underwriters' rules. The number of 40-: lamps and length of wire for each circuit are as follows:

Circuit *A*, 8 lights, 41 ft one way to load-center.
Circuit *B*, 11 lights, 52 ft one way to load-center.
Circuit *C*, 16 lights, 37 ft one way to load-center.
Circuit *D*, 12 lights, 59 ft one way to load-center.
Total number of lamps, 47.

·om Table V we see that the maximum length of line one way for No. 14 carrying twelve carbon or sixteen 40-watt lamps is 70 ft. Consequently, f the lamp-circuits can be No. 14 wire, which is the smallest size permitted.

Feed-Wires. These should be run on the three-wire system. Allowing for 2×47 or 94 lamps in first and second stories and eight in basement, the feed-wires must be capable of supplying 102 lamps. Each 40-watt lamp would take $40/110 = 0.364$ ampere. The distance from outside the building to distribution-cabinet is about 72 ft, allowing for three drops. Using Formula (1), and assuming that there will be fifty-one lamps on each side of the three-wire system, and doubling the drop in volts, gives

$$\text{Circular mils} = \frac{10.4 \times 144 \times 0.364 \times 51}{4} = 6\,960 \text{ c.m.}$$

which calls for No. 11 wire; but as this size is not carried in stock we must use No. 10. From the second story to the third No. 12 wires could be used. For almost all buildings lighted from a central station the lamp-circuits will not usually require a wire larger than No. 14, so that about the only wires which the architect needs to look after are the wires which run to the distribution-cabinets.

Switches. A switch is a device for opening or closing a circuit at will either at the fixture or at any other point. In the better class of buildings the majority, if not all, of the ceiling-lights are controlled by switches placed at a convenient place on a side wall. Lights may be controlled at any distance from the fixture by running a switch-loop. For controlling either a single lamp or fixture, or any number of lamps, a switch-loop is run as shown on circuits *A* and *C*, as in Fig. 13. As shown also in Fig. 4, one side of the loop must be connected with one of the distributing

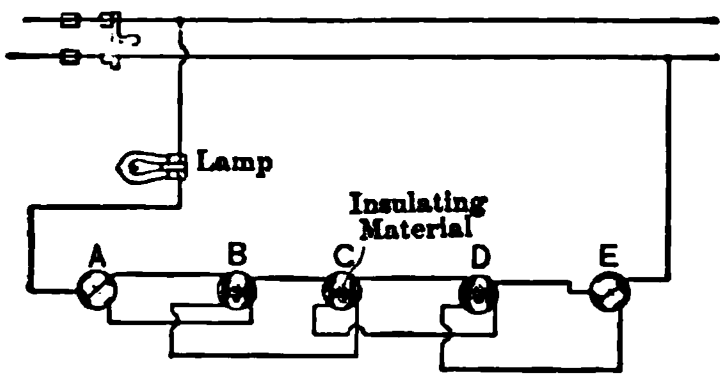


Fig. 15. The Lamp May Be Turned Off or On From Any of the Five Points, *A*, *B*, *C*, *D*, or *E*

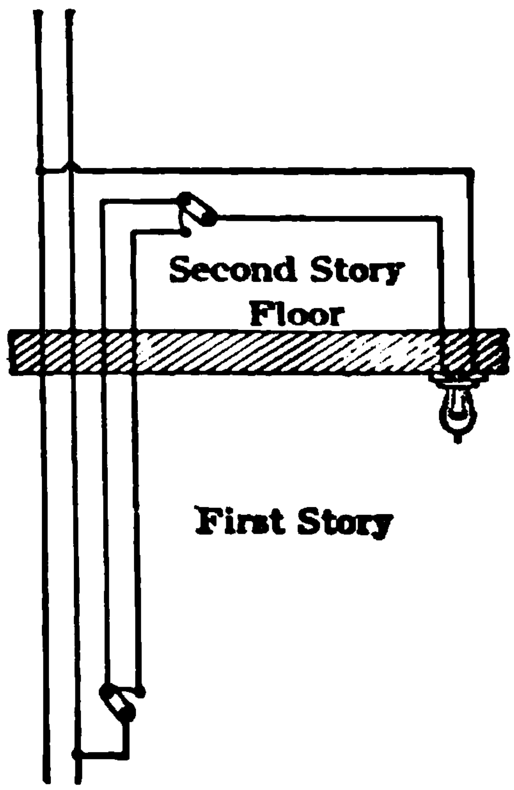


Fig. 16. The Lamps May Be Turned Off or On From Either the First or Second Story

wires and the other side to the lamp. When a number of lamps are to be controlled by one switch, as in the case of hall-lights, and the lamps in large rooms, such as churches, theaters, concert-halls, etc., a separate circuit is usually run for those lamps, and a switch anywhere in one of the distributing lines will turn on or off all of the lights on that line. As the underwriters do not permit more than twelve 16-candle-power carbon or sixteen 40-watt tungsten-lamps on one circuit, not more than these numbers of lamps can be controlled by one switch, except where the switch is placed on the mains. It is also practicable to control one lamp from two or three places. Thus by a duplex or three-point switch and proper wiring, a lamp may be lighted or turned off from either the first or second story at will. By means of two three-point switches and one four-point switch a first-story hall-lamp may be

controlled at will from either the first, second or third stories. Fig. 15 shows the method of control from any number of points, since any number of 4-point snap-switches, such as *B*, *C* and *D*, can be inserted between the 3-point switches *A* and *E* if more points of control are needed. Fig. 16 shows one method of wiring for controlling a hall-light from first and second stories by means of two 3-point switches. With the switches in the position shown the circuit is broken, as there is no connection between the lamps and line *B*. By turning either switch a connection is made with line *B* and the current will flow.

Kinds of Switches. For controlling lamps from one point three kinds of switches are used, namely, SNAP-SWITCHES, FLUSH or PUSH-BUTTON SWITCHES and KNIFE-SWITCHES. When less than eight lamps are controlled by the switch, flush or push button switch is commonly used where a neat appearance is desirable, and in places where this is of no importance, a snap-switch is used, as it is the cheaper. Where a circuit of twelve or more lamps is controlled by a switch, a double-pole (d p.) knife-switch (Fig. 17) is commonly used, being generally placed in a cabinet. Knife-switches should always be used on main wires. Snap and push-button switches are made both single and double pole. A SINGLE POLE switch opens only one side of the circuit and a DOUBLE-POLE switch both sides. A double-pole knife-switch necessarily opens both sides. A switch used on a three-wire system must have three poles. Double-pole snap and push button switches are seldom used for less than twelve lamps. DUPLEX SWITCHES, sometimes called THREE-POINT SWITCHES, are usually of the snap or of the push-button type.

Fig. 17. Common Knife-switch

Conduit-Systems. As weather-proof or rubber-covered wire cannot be run in brick walls or floors of brick, terra-cotta, or concrete without some protection other than the covering of the wires, it is necessary in such places to run the wires in tubes or conduits, and in fire-proof buildings all of the lighting-wires are generally run in a system of conduits.

Kinds of Conduits. There are two kinds of interior conduits now in common use

(1) Lined Mild-Steel Pipe. The lining consists of a thin coat of enamel which must be impervious to water, sulphuric acid, acetic acid, hydrochloric acid and carbonate-of-soda solutions. For regular conduit systems only mild-steel piping of the same thickness as ordinary gas-piping is approved by the Underwriters. The conduit must be continuous from outlet to outlet or junction-boxes or cabinets and must properly enter and be secured to all fittings, and the entire system must be mechanically secured in position. Mild-steel pipe may be galvanized, coated, or enameled on the outside, but it must be enameled on the inside as stated above Rigid conduit, WHETHER LINED OR UNLINED, are installed in the same manner as a good job of gas-fitting, except that for conduits the pipe may be bent to a curve and no elbow can be used having less than 3-in radius for the inner edge Wherever branches are taken off, junction-boxes must be provided and every outlet must have an approved outlet-box or plate. The wire drawn into conduits must be of at least No 14 size, rubber-covered and with double braid. All conduit-systems must be GROUNDED by connecting the steel pipe by a conductor to the gas or water system.

(2) **Flexible Armored Conduit.** This is made of metal ribbon wound spirally, is generally used in wiring old houses because it is easier to install. CIRCULAR LOOM is flexible woven tubing treated with insulating material that makes it hold its shape. This may be used in dry places and for outlets through plastering if it extends back to the nearest porcelain knob holding the wire which the conduit covers.

National Electrical Code. The National Board of Fire Underwriters, in conjunction with committees from the American Institute of Architects, and from the national associations of electrical, mechanical and railway engineers, have prepared a code of rules and requirements for the installation of electrical lighting which is the generally recognized standard and with which all interior wiring must comply if it is desired to obtain insurance on the building. This code has also been made a part of the ordinances of most of the larger cities. It is revised every two years, in the odd-numbered years. The National Board of Underwriters also publishes, semi-annually, a SUPPLEMENT to the National Electrical Code which contains a list of all articles that have been examined and approved for use in connection with the code, together with the names of the manufacturers. Articles not included in this list will not be passed by the inspectors. Copies of the code and supplement can be obtained from the nearest Underwriters' Inspection Bureau, or by writing to the Underwriters' Laboratories, 382 Ohio Street, Chicago, Ill. The following requirements apply to almost every installation, and every architect should be conversant with them.

Extracts from the National Electrical Code*

(1) All wire for concealed work must be of the best approved rubber-covered brands, as shown in List of Fittings. No wire smaller than No. 14 B. & S. gauge to be used. All wire run in conduits must have double-braid covering.

(2) Where wires are concealed and run parallel to joists they must be supported on porcelain knobs which hold the wires at least 1 in from woodwork or surface wired over. Knobs must be SECURELY FASTENED and MUST BE PLACED EVERY 4½ FT. Where wires are run through joists they must be bushed with porcelain tubes the entire width of joists. All wires must be drawn tight, so as to have all slack removed.

(3) In concealed work all wires MUST BE SEPARATED FROM EACH OTHER BY AT LEAST 5 IN. Where wires run down partitions, especially partitions formed by 2 by 4-in studs, the wires must be so supported as to run in the middle of partition. If more than two wires are run down partition between studs, they must be separated by at least 5 in.

(4) Where wires pass through floors they must be protected from the floor up to a point 5 ft above the floor with conduit or with boxing. There must always be a space of 1 in between the wires and the boxing.

(5) All joints must be securely soldered and taped. A splice to be approved must be both mechanically and electrically secure without solder, but must be soldered unless made with some form of APPROVED splicing-device. Joints to be properly taped require, where rubber-covered wire is used, first to be taped with rubber tape and then with friction-tape. The insulation of a joint must equal that on the conductors.

(6) Where wires enter the building they must be provided with drip-loops.

(7) There must be a MAIN CUT-OUT AND SWITCH installed in an easily accessible place, as near as possible to the point where the wires enter the building.

* The numbers here given do not correspond with those in the code, and several of the rules are much abridged. They are intended to give the substance, rather than the exact language.

This will require that cut-out and switch be placed where there is no need of a 12-ft ladder to reach them. (See Fig. 18.)

(8) Every lighting-circuit of 660 watts must be protected by a cut-out. This will limit the number to twelve 16-candle-power or sixteen 40-watt lights on a two-wire, 110-volt circuit, and to thirty-two 40-watt or twenty 16-candle-power lights on a three-wire, 220-volt circuit. By special permission, where No. 14 wire is carried directly to keyless sockets, and where the location of the sockets s such as to render unlikely the attachment of flexible cords thereto, the circuits nay be so arranged that not more than 1 320 watts (or 32 sockets) may be de- endent upon the final cut-out. Sockets are to be considered as requiring not ess than 40 watts each.

(9) All cut-outs must be placed in an ASBESTOS-LINED CABINET. The as- estos must be at least 1/8 in in thickness and securely held in place by shellac nd tacks. Lumber of which cabinet is made must be at least 3/4 in in thickness. Cabinet must be furnished with snug-fitting door; door to be hung by strong inges and to be furnished with a suitable catch.

(10) Cut-outs to be approved must be of the plug or of the cartridge-type.

(11) Enclosed arc-lamps and incandescent lamps must not be placed on same rcuit. Arcs must be on separate circuits by themselves. Each arc-light must e protected by an approved cut-out. The cut-outs are to be placed in an as- estos-lined cabinet.

(12) The practice of using fused rosettes will not be approved, except in mills.

(13) Where wires run down the side wall they must be protected from me- anical injury.

(14) All outlets must be made to conform to Rule 24, National Electrical Code.

(15) Fans in series will not be approved.

(16) Runs of lamp-cord will not be approved. Lamp-cord is designed to be ed for drops only. Ordinary insulated wire must be run to place desired.

(17) Electric heaters must be installed in accordance with Rule 25 a–f, National ectrical Code.

General Suggestions for Electric Work *

General Principles and Recommendations. In all electric-work con- ctors, however well insulated, should always be treated as bare, to the end it under no conditions, sting or likely to exist, a grounding or short cuit occur, and so that leakage from con- tor to conductor, or ween conductor and und, may be reduced the minimum. In wiring special atten- must be paid to the mechanical execution of the work. Careful and t running, connecting, soldering, taping of conductors, and securing and ching of fittings, are specially conducive to security and efficiency, and will strongly insisted on. In laying out an installation, except for constant-cur- systems, the work should, if possible, be started from a center of distri- ion, and the switches and cut-outs, controlling and connected with the several iches, be grouped together in a safe and easily accessible place, where they be readily got at for attention or repairs. The load should be divided as

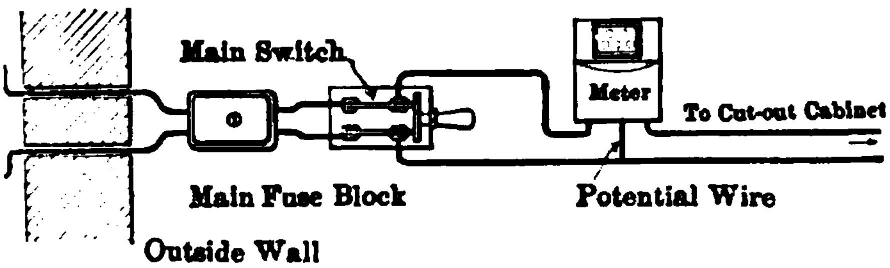


Fig. 18. Main Switch, Fuse-block and Meter Located Near the Point of Entrance of the Service-wires

* Preface to the National Electrical Code

evenly as possible among the branches, and all complicated and unnecessary wiring avoided. The use of wireways for rendering concealed wiring permanently accessible is most heartily indorsed and recommended; and this method of accessible concealed construction is advised for general use. Architects are urged, when drawing plans and specifications, to make provision for the channeling and pocketing of buildings for electric-light or power-wires, and in specifications for electric gas-lighting to require a two-wire circuit, whether the building is to be wired for electric lighting or not, so that no part of the gas-fixtures or gas-piping be allowed to be used for the gas-lighting circuit. Fig. 18 shows a common arrangement of main cut-out, switch and meter, to comply with Rule 7, page 1480. The main cut-out and switch should be as near as possible to the outside wall, but the meter may be at some distance from the switch if desirable for any reason.

Specifications for Interior Wiring*

Specifications for Interior Wiring should provide:

(1) That the wiring shall be installed in accordance with the latest rules and requirements of the National Board of Fire Underwriters, the local ordinances, and the rules of the local electric light company, where current is to be taken from the public mains.

(2) No electrical device or material of any kind to be used that is not approved by the Underwriters' National Electric Association, and all articles must have the name or trade-mark of the manufacturer and the rating in volts and amperes or other proper units marked where they may readily be observed after the device is installed.

Requirements (1) and (2) are sufficient to insure a SAFE installation.

(3) Contractor must obtain a satisfactory certificate of inspection from the city inspector or from the inspector of the local board of fire-underwriters.

(4) If the wires are to run in a conduit system it should be so specified. When a conduit system is used, THE WIRES SHOULD NOT BE DRAWN IN until all mechanical work as far as possible is completed. It is best to wait until after the plastering is dry. All conduit systems must be GROUNDED.

(5) Size of Wires. The best method is to specify the size of all wires, no wire to be less than No. 14 B. & S. gauge; but if the architect does not care to do this, the following clause is sufficient, provided he can have confidence that the contractor will comply with it: "All wires must be of such size that the drop in potential at farthest light-outlet shall not exceed 2% under maximum load."

(6) Cut-out cabinets and where they are to be placed; also location of main-line cut-out and fuse. For buildings containing not more than forty lights, one distributing point is generally sufficient, although in large houses it is often convenient to have a cut-out cabinet in each story.

(7) Number and kind of switches. All outlets should be marked on the plans, and the number of lights indicated by figures 1, 2, 3, 4, etc., as in Fig. 13. See pages 1484 and 1485 for standard symbols. The location of all switches for controlling lights should also be indicated on the plans.

Approximate Cost of Wiring for Incandescent Lighting. Approximate estimates of the cost of wiring buildings for electric lighting are usually based on the number of outlets (not lamps). The actual cost will depend upon the number of pounds of wire required, the kind and number of switches, character of cut-out cabinets, etc., and the time required to do the work, so that a close

* Wiring specifications for buildings having their own generating plant should be prepared by an expert.

estimate cannot be made without plans and specifications. Again, wages and prices of material vary to a considerable extent in different parts of the country, so that an estimate that would be about right for one locality would not suffice for another. The following figures,* however, will enable anyone to form an approximate idea of what any proposed wiring-job will cost.

Count cost of labor as not more than one-third the cost of the installation.

For knob-and-tube work in new houses of less than seventeen outlets or twenty-five lamps, with no switches except main switch and a rough cut-out box lined with asbestos, allow $1.50 per outlet.

For same class of work, from 25 to 100 lamps, allow $1.75 to $2.00 per outlet.

The extra labor involved in wiring old buildings will add from 30 to 50% to the above figures.

For each switch-loop with a single-pole snap-switch, add from $1.50 to $1.75.

For each switch-loop with single-pole push-button switch, add from $2.25 to $2.50.

For each lamp controlled by duplex or three-point switches, add from $5 to $6.

For each hardwood cut-out cabinet with door and lock, add from $7 up according to number of circuits and finish.

Iron cut-out cabinets cost from $8.50 up.

Ordinary exposed wiring, as in factories, can usually be run for from $1.00 to $1.75 per drop, including rosettes, cord and sockets, the cost depending very largely upon how closely the drops are spaced.

Small installations with iron-armored conduit will probably cost from $5 to $6 per outlet. Large installations will cost somewhat less.

A private lighting-plant of 200 lamps, wired on the concealed knob-and-tube system, will cost from $1 250 to $1 500, and a similar plant with 600 lamps will cost from $2 500 to $3 000. These prices include engine, dynamo-switchboard, etc., complete, and wiring, but no switches for controlling lamps.

The iron-armored conduit-system will add about $2.75 per outlet.

None of the above estimates include the cost of fixtures except in the case of exposed wiring.

Drop-cord and sockets cost about 90 cts per lamp. Single-lamp fixtures may be purchased from $1.25 upwards; double-lamp fixtures from $2 upwards. Combination-fixtures cost about 25% more than straight electric fixtures.

The price of rubber-covered wire varies from $8 to $60 per 1 000 ft according to size, and of weather-proof wire from 16 cts to 25 cts per pound.

* These are pre-war prices and the data are retained for purposes of comparison of relative values.

Standard Wiring-Symbols Adopted by the National Contractors' Association and the American Institute of Architects

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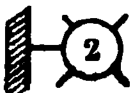
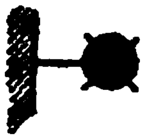
 Ceiling-outlet; electric only. Numeral in center indicates number of standard 16-c.p. incandescent lamps.*

 $\frac{4}{2}$ Ceiling-outlet; combination. $\frac{4}{2}$ indicates 4–16c.p. standard incandescent lamps and 2 gas-burners. If gas only. 

 Bracket-outlet; electric only. Numeral in center indicates number of standard 16-c.p. incandescent lamps.

 $\frac{4}{2}$ Bracket-outlet; combination. $\frac{4}{2}$ indicates 4–16 c.p. standard incandescent lamps and 2 gas-burners. If gas only. 

 Wall or baseboard receptacle-outlet. Numeral in center indicates number of standard 16-c.p. incandescent lamps.

 Floor-outlet. Numeral in center indicates number of Standard 16-c.p. incandescent lamps.

 6 Outlet for outdoor standard or pedestal, electric only. Numeral indicates number of standard 16-c.p. incandescent lamps.

 $\frac{6}{6}$ Outlet for outdoor standard or pedestal; combination. $\frac{6}{6}$ indicates 6–16 c.p. standard incandescent lamps; 6 gas-burners.

 Drop-cord outlet.

 One-lamp outlet, for lamp-receptacle.

 Arc-lamp outlet.

 Special outlet for lighting, heating and power-current, as described in specifications.

 Ceiling-fan outlet.

S^1 S. P. switch-outlet.

S^2 D.P. switch-outlets.

S^3 3-way switch-outlet.

S^4 4-way switch-outlet.

S^D Automatic door switch-outlet.

S^E Electrolier switch-outlet.

Show as many symbols as there are switches. Or in case of a very large group of switches, indicate number of switches by a Roman numeral, thus; S' XII, meaning 12 single-pole switches.

Describe type of switch in specifications, that is, flush or surface, push-button or snap.

 Meter-outlet.

 Distribution-panel.

 Junction or pull-box.

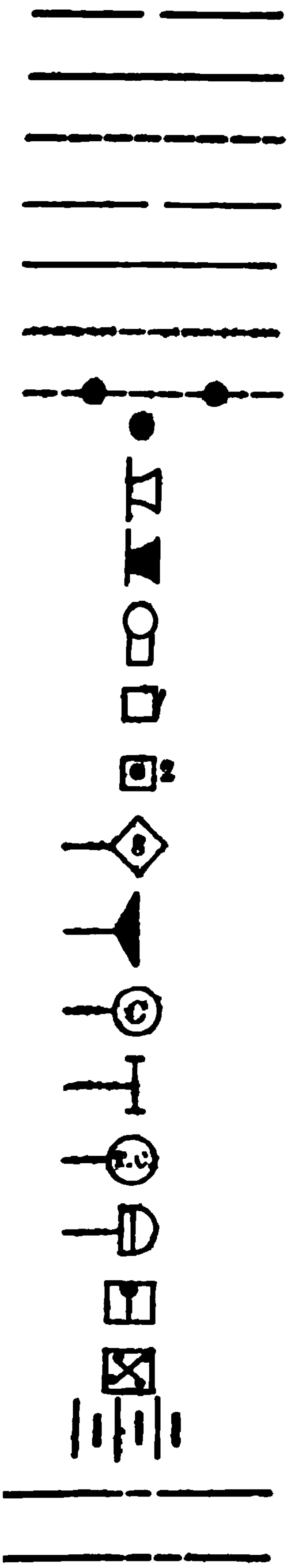
 Motor-outlet. Numeral in center indicates horse-power.

 Motor-control outlet.

 Transformer.

* If tungsten-lamps are used instead of carbon-lamps, the figure in the circle may stand for the number of 25-watt tungsten-lamps, a 25-watt tungsten-lamp being the nearest in candle-power to a 16-candle-power carbon-lamp though consuming less than one-half the power. Since tungsten-lamps average about 1.1 watts to the candle-power, many architects place in the circle the number of watts to be used. Dividing this number

Standard Wiring-Symbols Adopted by the National Contractors' Association and the American Institute of Architects (Continued)



Main or feeder-run concealed under floor.

Main or feeder-run concealed under floor above.

Main or feeder-run exposed.

Branch circuit-run concealed under floor.

Branch circuit-run concealed under floor above.

Branch circuit-run exposed.

Pole-line.

Riser.

Telephone-outlet; private service.

Telephone-outlet; public service.

Bell-outlet.

Buzzer-outlet.

Push-button outlet. Numeral indicates number of pushes.

Annunciator. Numeral indicates number of points.

Speaking-tube.

Watchman-clock outlet.

Watchman-station outlet.

Master time-clock outlet.

Secondary time-clock outlet.

Door-opener.

Special outlet for signal-systems, as described in specifications.

Battery-outlet.

Circuit for clock, telephone, bell or other service, run under floor, concealed. Kind of service wanted ascertained by symbol to which line connects.

Circuit for clock, telephone, bell or other service, run under floor above, concealed. Kind of service wanted ascertained by symbol to which line connects.

ights of center of wall-outlets (unless otherwise specified):

Living-rooms	5 ft 6 in
Chambers	5 ft 0 in
Offices	6 ft 0 in
Corridors	6 ft 3 in
ights of switches (unless otherwise specified)	4 ft 0 in

1.1 gives the candle-power per outlet. Thus (120) means enough tungsten-lamps be placed in this outlet to total 120 watts, three 40-watt lamps, or two 60-watt ps, etc. The candle-power in any case would be 120/1.1 = 110 candle-power.

ARCHITECTURAL ACOUSTICS*

By

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Architectural Acoustics a Rational Engineering Problem. Because familiarity with the phenomena of sound has so far outstripped the adequate study of the problems involved, many of them have been popularly shrouded in a wholly unnecessary mystery. Of none, perhaps, is this more true than of ARCHITECTURAL ACOUSTICS. The conditions surrounding the transmission of speech in an enclosed auditorium are complicated, it is true, but are only such as will yield an exact solution in the light of adequate data. It is not unreasonable, therefore, to include problems of architectural acoustics among the RATIONAL ENGINEERING PROBLEMS.

Character and Application of the Problem. The problem of architectural acoustics is necessarily complex, and each room presents many conditions which contribute to the result in a greater or less degree, according to circumstances. To take justly into account these varied conditions, the solution of the problem should be QUANTITATIVE, not merely QUALITATIVE; and to reach its highest usefulness and the dignity of an engineering science it should be such that its application can precede, not merely follow, the construction of the building.

Conditions and Factors of the Problem. In order that hearing may be good in any auditorium it is necessary that the sound should be sufficiently loud, that the simultaneous components of a complex sound should maintain their proper relative intensities, and that the successive sounds in rapidly moving articulation, either of speech or of music, should be clear and distinct, free from each other and from extraneous noises. These three are the necessary, as they are the entirely sufficient, conditions for good hearing. Scientifically the problem involves three factors:

(1) Reverberation.
(2) Interference.
(3) Resonance.

As an engineering problem it involves the shape of the auditorium, its dimensions, and the materials of which it is composed.

Rate of Absorption of Sound. Sound, being energy, once produced in a confined space, will continue until it is either transmitted by the boundary walls or is transformed into some other kind of energy, generally heat. This process of decay is called ABSORPTION. Thus, in the lecture-room of Harvard University, in which, and in behalf of which, this investigation was begun, the RATE

* Adapted and reproduced by permission from a paper read by Dr. W. C. Sabine before the Franklin Institute, Philadelphia, October 30, 1914 and published in the January 1915 issue of the Journal of the Franklin Institute. For information regarding further data, see other papers and treatises on the subject by the author of this article.

OF ABSORPTION was so small that a word spoken in an ordinary tone of voice was audible for five and a half seconds afterwards. During this time even a very deliberate speaker would have uttered the twelve or fifteen succeeding syllables. Thus the successive enunciations blended into a loud sound, through which and above which it was necessary to hear and distinguish the orderly progression of the speech. Across the room this could not be done; even near the speaker it could be done only with an effort wearisome in the extreme if long maintained.

Multiple Reflection, Reverberation and Echoes. With an audience filling the room the conditions were not so bad, but still not tolerable. This may be regarded, if one so chooses, as a process of MULTIPLE REFLECTION from walls, rom ceiling, and from floor, first from one and then another, losing a little at ach reflection until ultimately inaudible. This phenomenon will be called REVERBERATION, including, as a special case, the ECHO. It must be observed, owever, that, in general, reverberation results in a mass of sound filling the whole room and incapable of analysis into its distinct reflections. It is thus more lifficult to recognize and impossible to locate. The term ECHO will be reserved or that particular case in which a short, sharp sound is distinctly repeated by eflection, either once from a single surface, or several times from two or more urfaces.

Rate of Decay of Sound. In the general case of reverberation we are oncerned only with the RATE OF DECAY of the sound. In the special case of he echo we are concerned not merely with its intensity, but with the interval f time elapsing between the initial sound and the moment it reaches the observer. In the room mentioned as the occasion of this investigation no discrete cho was distinctly perceptible, and the case will serve excellently as an illustraion of the more general type of reverberation.

Duration of Audibility of Residual Sound. After preliminary gropings, rst in the literature and then with several optical devices for measuring the itensity of sound, all established methods were abandoned. Instead, the RATE F DECAY was measured by measuring what was inversely proportional to it, he duration of audibility of the reverberation, or, as it will be called here, the URATION OF AUDIBILITY OF THE RESIDUAL SOUND. These experiments may be xplained to advantage here, for they will give more clearly than would abstract iscussion an idea of the nature of reverberation.

Shape of Room and Nature of Furnishings. Broadly considered there re two, and only two, variables in a room, SHAPE (including size), and MATERIALS ncluding furnishings). In designing an auditorium an architect can give condeation to both; in repair-work for bad acoustic conditions it is generally npracticable to change the shape, and only variations in materials and irnishings are allowable. This was, therefore, the line of work in this ise.

The Relative Absorbing Power of Different Substances. It was evident iat, other things being equal, the rate at which the reverberation would disppear was proportional to the rate at which the sound was absorbed. The rst work, therefore, was to determine the RELATIVE ABSORBING POWER of various ibstances. With an organ-pipe as a constant source of sound, and a suitable ironograph for recording, the duration of audibility of a sound after the source id ceased in this room when empty was found to be 5.62 seconds. All the ishions from the seats in Sanders Theater, Boston, Mass., were then brought ver and stored in the lobby. On bringing into the lecture-room a number

of cushions, having a total length of 8.2 meters, the duration of audibility fell to 5.33 seconds. On bringing in cushions of a total length of 17 meters the sound in the room after the organ-pipe ceased was audible for but 4.94 seconds. Evidently the cushions were strong absorbents and rapidly improving the room, at least to the extent of diminishing the reverberation. The result was interesting and the process was continued. Little by little more cushions were brought into the room, and each time the duration of audibility was measured. When all the seats, 436 in number, were covered, the sound was audible for 2.03 seconds. Then the aisles were covered, and then the platform. Still there were more cushions, almost half as many more. These were brought into the room, a few at a time, as before, and draped on a scaffolding that had been erected around the room, the duration of the sound being recorded each time. Finally, when all the cushions from a theater seating nearly 1 500 persons were placed in the room, covering the seats, the aisles, the platform, and the rear wall to the ceiling, the duration of audibility of the residual sound was 1.14 seconds. This experiment, requiring, of course, several nights' work, having been completed, all the cushions were removed and the room was in readiness for the test of other absorbents. It was evident that a STANDARD OF COMPARISON had been established. Curtains of chenille, 1.1 meters wide and 17 meters in total length, were draped in the room. The duration of audibility was then 4.51 seconds. Turning to the data that had just been collected, it appeared that this amount of chenille was equivalent to 30 meters of cushions from Sanders Theater. Oriental rugs (Herez, Demirjik, and Hindoostanee) were tested in a similar manner, as were also cretonne cloth, canvas, and hair-felt. Similar experiments, but in a smaller room, determined the absorbing power of a man and of a woman, always by determining the number of running meters of Sanders Theater cushions that would produce the same effect. This process of comparing two absorbents by actually substituting one for the other is laborious, and it is given here only to show the first steps in the development of a method. Without going into details, it is sufficient here to say that this method was so perfected as to give not merely RELATIVE, but ABSOLUTE, COEFFICIENTS OF ABSORPTION.

Coefficients of Absorption. In this manner a number of COEFFICIENTS OF ABSORPTION were determined for objects and materials which could be brought into and removed from the room, for sounds having a pitch an octave above middle C. In the following table the numerical values are the ABSOLUTE COEFFICIENTS OF ABSORPTION:

Oil-paintings, inclusive of frames	0.28
Carpet-rugs	0.20
Oriental rugs, extra heavy	0.29
Cheese-cloth	0.019
Cretonne cloth	0.15
Shelia curtains	0.23
Hair-felt, 2.5 cm. thick, 8 cm. from wall	0.78
Cork, 2.5 cm. thick, loose on floor	0.16
Linoleum, loose on floor	0.12

When the objects are not extended surfaces, such as carpets or rugs, but essentially spacial units, it is not easy to express the absorption as an absolute coefficient. In the following table the absorption of each object is expressed in terms of a SQUARE METER OF COMPLETE ABSORPTION:

Audience, per person	0.44
Isolated woman	0.54
Isolated man	0.48
Plain, ash settees	0.039
Plain, ash settees, per single seat	0.0077
Plain, ash chairs, bent wood	0.0082
Upholstered settees, hair and leather	1.10
Upholstered settees, per single seat	0.28
Upholstered chairs, similar in style	0.30
Hair-cushions, per seat	0.21
Elastic-felt cushions, per seat	0.20

Coefficient of Absorption of Floors, Ceilings and Wall-Surfaces. Of ven greater importance was the determination of the COEFFICIENT OF ABSORP-ON of floors, ceilings, and all-surfaces. The accom-ishment of this called for a ery considerable extension the method adopted. If e reverberation in a room changed by the addition absorbing material are otted, the resulting curve ll be found to be a portion a hyperbola with dis-aced axes. An example of ch a curve, as obtained in e lecture-room of the Fogg t Museum, Cambridge, ass., is plotted in the agram in Fig. 1. If now e origin of this curve is splaced so that the axes of coordinates are the asymptotes of the rectangular perbola (Fig. 2), the displacement of the origin measures the initial absorbing wer of the room, its floors, walls and ceilings. Such experiments were carried t in a large number of rooms in which the different component materials tered in very different degrees, and an elimination between these different periments gave the following COEFFICIENT OF ABSORPTION for different aterials:

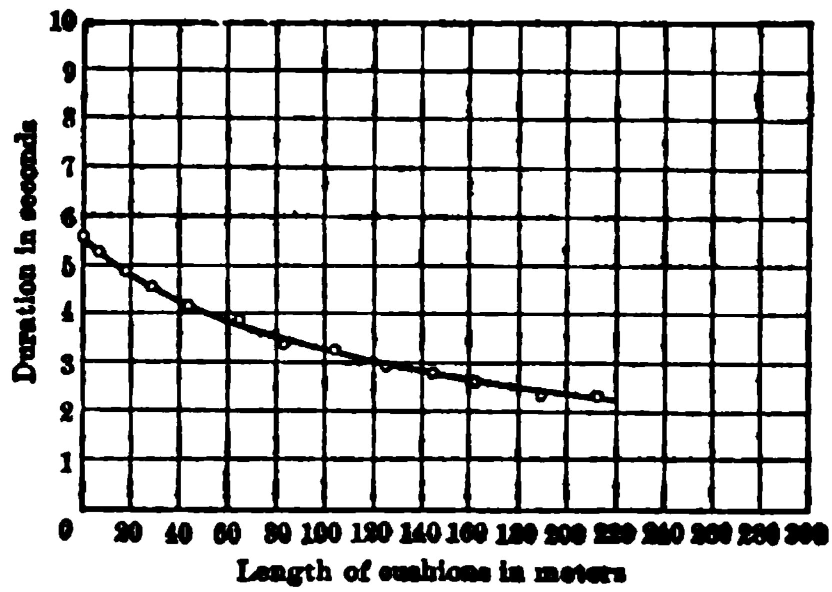


Fig. 1. Curve Showing the Relation of Duration of Residual Sound to Added Absorbing Material

Open window	1.000
Wooden sheathing, hard pine	0.061
Plaster on wooden lath	0.034
Plaster on wire lath	0.033
Glass, single thickness	0.027
Plaster on tile	0.025
Brick set in Portland cement	0.025

Calculating the Reverberation for Any Room. If the experiments in se rooms are plotted in a single diagram, the result is a family of HYPERBOLAS ig. 3) showing a very interesting relationship to the volumes of the rooms. deed, if from these hyperbolas the parameter, which equals the product of the

coordinates, is determined, it will be found to be linearly proportional to the volume of the room. These results are plotted in Fig. 4, showing how strict the proportionality is even over a very great range in volume. We have thus at hand a ready method of calculating the REVERBERATION for any room, its volume and the materials of which it is composed being known. The first five years of the investigation were devoted to violin C, the C an octave above middle C, having a VIBRATION-FREQUENCY of 512 vibrations per second. This pitch was chosen because, in the art of telephony, it was regarded at that time as the characteristic pitch determining the conditions of articulate speech. The planning of Symphony Hall, Boston, Mass., forced an extension of this investigation to notes over the whole range of the musical scale, three octaves below and three octaves above violin-C.

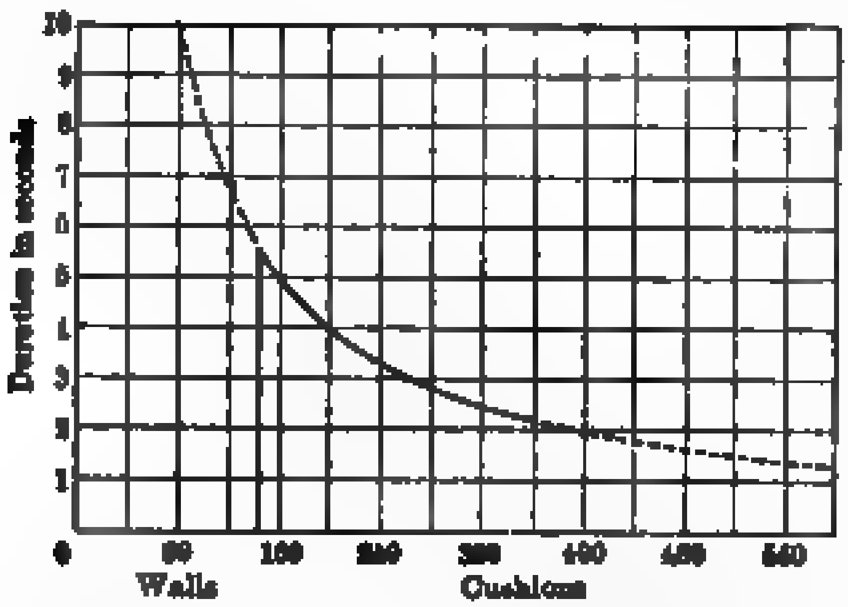


Fig. 2. Curve 5 Plotted as Part of its Corresponding Rectangular Hyperbola. The Solid Part was Determined Experimentally. The Displacement of This to the Right Measures the Absorbing Power of the Walls of the Room

Absorption-Coefficient of an Audience. In the very nature of the problem, the most important datum is the ABSORPTION-COEFFICIENT of an audience, and the determination of this was the first task undertaken. By means of a lecture on one of the recent developments of physics, wireless telegraphy, an audience was thus drawn together and at the end of the lecture requested to remain for the experiment. In this attempt the effort was made to determine the coefficients for the five octaves from $C_2 128$ to $C_6 2048$, including notes E and G in each octave. For several reasons the experiment was not a success. A threatening thunderstorm made the audience a small one, and the sultriness of the atmosphere made open windows necessary; while the attempt to cover so many notes, thirteen in all, prolonged the experiment beyond the endurance of the audience. While this experiment failed, another, the following summer, was more successful. In the year that had elapsed the necessity of carrying the investigation further than the limits intended became evident, and now the experiment was carried from $C_1 64$ to

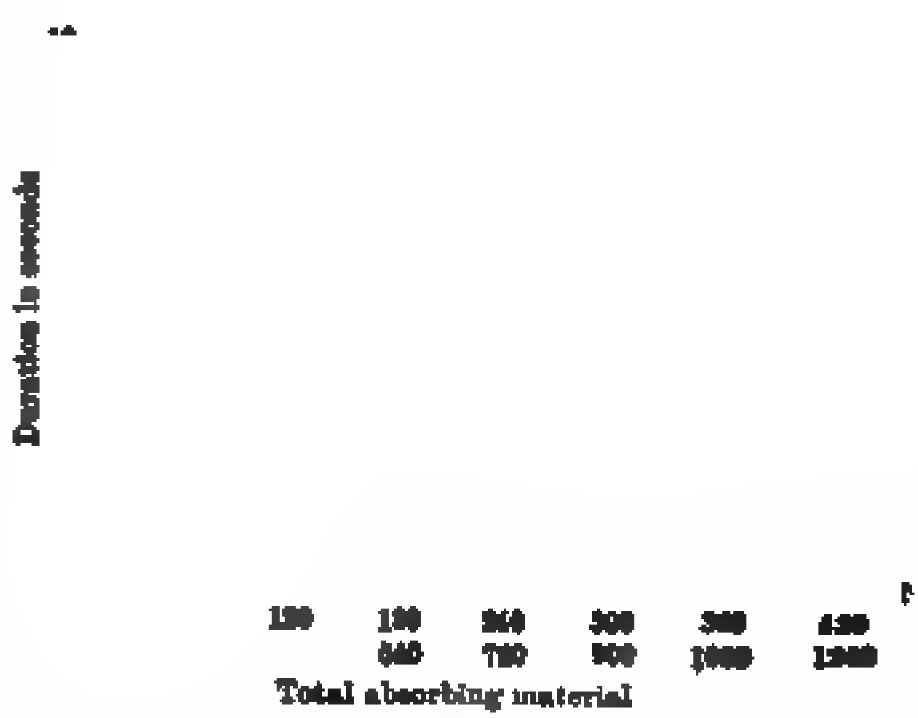


Fig. 3. Curves Entered as Parts of their Corresponding Rectangular Hyperbolas. Three Scales are Employed for the Volumes, by Groups, 1–7, 8–11, and 12

$C_4 4096$, but included only the C notes, seven notes in all. Moreover, bearing in mind the experiences of the previous summer, it was recognized that even seven notes would come dangerously near overtaxing the patience of the audience. Inasmuch as the COEFFICIENT OF ABSORPTION for $C_4 512$ had already been determined six years before, in the investigations mentioned, the coefficient for this note was not redetermined. The experiment was therefore carried out for the lower three and the upper three notes of the seven. The audience, on the night of this experiment, was much larger than that which came the previous summer, the night was a more com-fortable one, and it was

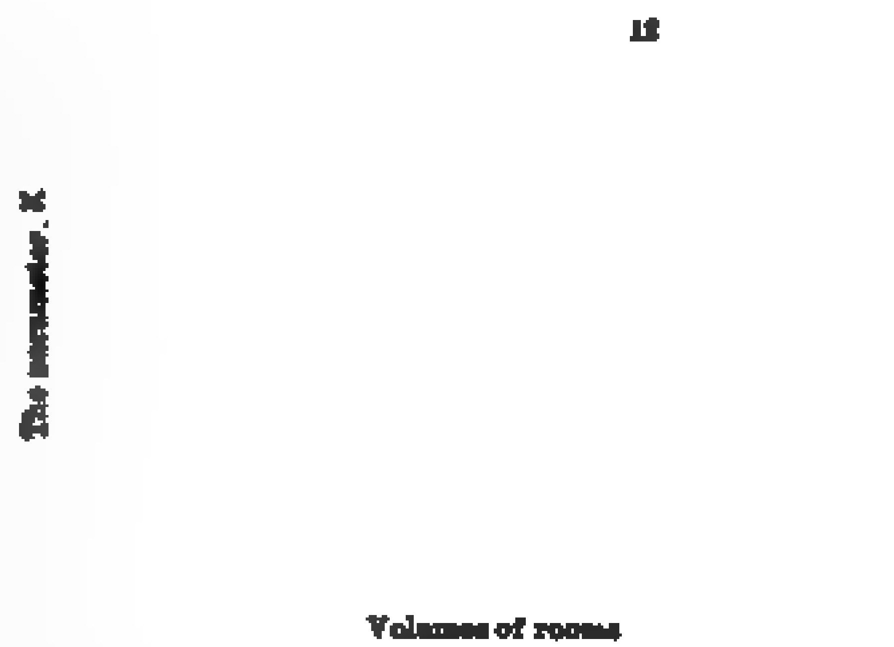
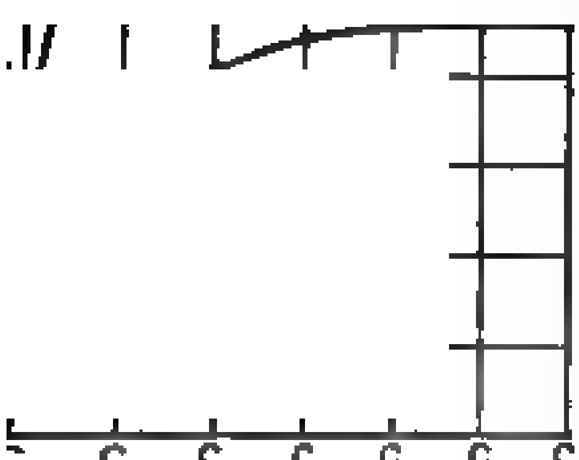


Fig. 4. The Parameters k, Plotted Against the Volumes of the Rooms, Showing the Two Proportional

possible to close the windows during the experiment. The conditions were hus fairly satisfactory. In order to get as much data as possible, and in as hort a time, there were nine observers stationed at different points in the room. These observers, whose kindness and skill it is a pleasure to acknowledge, had prepared themselves, by previous practice, for this one experiment. The results of the experiment are shown on the lower curve in Fig 5. This curve gives the COEFFICIENT OF ABSORPTION PER PERSON. It is to be observed that one of the points falls clearly off the smooth curve drawn through the other points. The observations on which this point is based were, however, much disturbed by a street-car passing not far from the building, and the departure of this observation from the curve does not indicate a real departure in the coefficient, nor should it cast much doubt on the rest of the work, in view of the circumstances under which it was secured. Counteracting the, perhaps, bad impression which this point may give, it is considerable satisfaction to note how accurately the point for $C_4 512$, determined six years before by a different set of observers, falls on the smooth curve through the remaining points. The upper curve represents the absorbing power of an audience per square meter, as ordinarily



. 5. Absorbing Power of an Audience for Different Notes

ted. The vertical ordinates are expressed in terms of total absorption a square meter of surface. For the upper curve the ordinates are thus the inary coefficients of absorption. The several notes are at octave-intervals

as follows: $C_1$64, $C_2$128, C_3 (middle C) 256, $C_4$512, $C_5$1024, $C_6$2048, $C_7$4096. In the audience on which these observations were taken there were 77 women and 105 men. The courtesy of the audience in remaining for the experiment and the really remarkable silence which they maintained are gratefully acknowledged.

Absorption of Sound by Wooden Sheathing. The next experiment was on the determination of the ABSORPTION OF SOUND by wooden sheathing. It is not an easy matter to find conditions suitable for this experiment. The room in which the absorption by wooden sheathing was determined in the earlier experiments was not available for these. It was available then only because the building was new and empty. When these more elaborate experiments were under way the room became occupied, and in a manner that did not admit of its being cleared. Quite a little searching in the neighborhood of Boston failed to discover an entirely suitable room. The best one available adjoined a night-lunch room. The night-lunch was bought out for a couple of nights, and the experiment was tried. The work of both nights was much disturbed. The traffic past the building did not stop until nearly two o'clock, and began again at four. The interest of those passing on foot throughout the night, and the necessity of repeated explanations to the police, greatly interfered with the work. This detailed statement of the conditions under which the experiment was tried is made by way of explanation of the irregularity of the observations recorded on the curve, and of the failure to carry this particular line of work further. On the first night seven points were obtained for the seven notes $C_1$64 to $C_7$4096. The reduction of these results on the following day showed variations indicative of maxima and minima, which, to be accurately located, would require the determination of intermediate points. In the experiment on the following night, points were determined for the E and G notes in each octave between $C_2$128 and $C_6$2048. Other points would have been determined, but time did not permit. It is obvious that the intermediate points in the lower and in the higher octave were desirable, but no pipes were to be had on such short notice for this part of the range, and in their absence the data could not be obtained. In the diagram, Fig. 6, the points lying on the vertical lines were determined the first night. The points lying between the vertical lines were determined the second night. The sheathing of the room is of North Carolina pine, 2 centimeters thick. The absorption is here due almost wholly to yielding of the sheathing as a whole. It is not possible now to learn as much in regard to the framing and arrangement of the studding in the particular room tested as is desirable. The accuracy with which these points fall on a smooth curve is, perhaps, all that could be expected in view of the difficulty under which the ob-

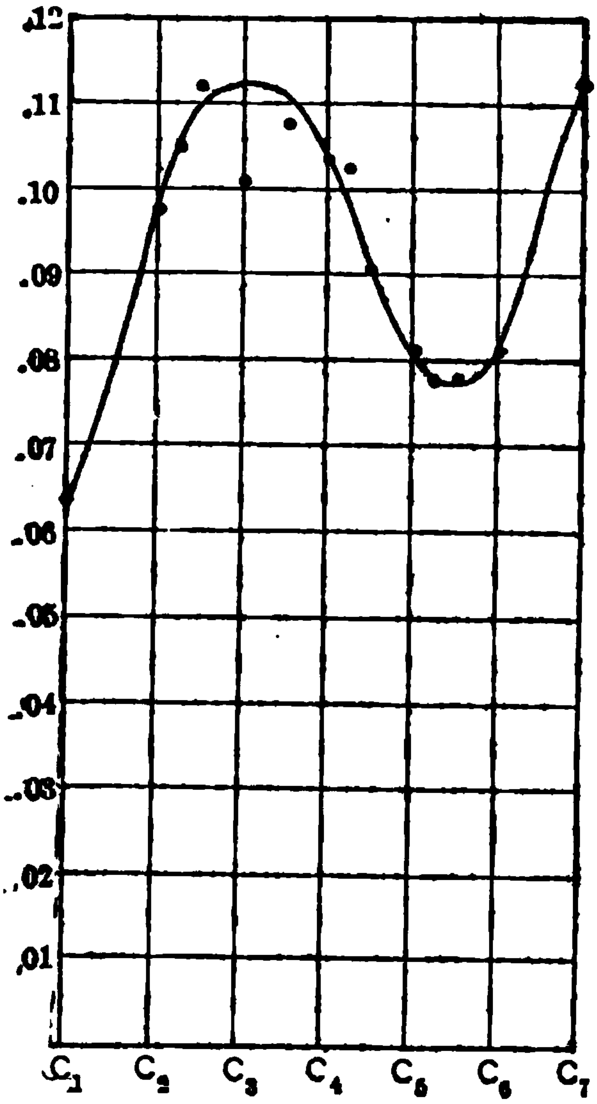


Fig. 6. Absorbing Power of Wooden Sheathing

servations were conducted and the limited time available. One point in particular falls far off from this curve, the point for C_3256, by an amount which is, to say the least, serious, and which can be justified only by the conditions under which the work was done. The general trend of the curve seems, however, established beyond reasonable doubt. It is interesting to note that there is one point of MAXIMUM ABSORPTION, which is due to resonance between the walls and the sound, and that this point of maximum absorption lies in the lower part, though not in the lowest part, of the RANGE OF PITCH tested. It would have been interesting to determine, had the time and facilities permitted, the shape of the curve beyond C_74096, and to see if it rises indefinitely, or shows, as is far more likely, a succession of maxima.

Absorption of Sound by Cushions. The experiment was then directed to the determination of the ABSORPTION OF SOUND by cushions, and for this purpose return was made to the constant-temperature room. Working in the manner indicated in the earlier papers for substances which could be carried in and out of a room, the curves represented in Fig. 7 were obtained. Curve 1 shows the ABSORPTION-COEFFICIENT for the Sanders Theater cushions, with which the whole investigation was begun ten years ago (1904). These cushions were of a particularly open grade of packing, a sort of dry grass or vegetable fiber. They were covered with canvas ticking, and that, in turn, with a very thin, cloth covering. Curve 2 is for cushions borrowed from the Phillips Brooks House. They were of high grade, filled with long, curly hair, and covered with canvas ticking, which was, in turn, covered by a long-nap plush. Curve 3 is for the cushion of Appleton Chapel, hair-covered with a leatherette, and showing a sharper maximum and a more rapid diminution in absorption for the higher frequencies, as would be expected under such conditions. Curve 4

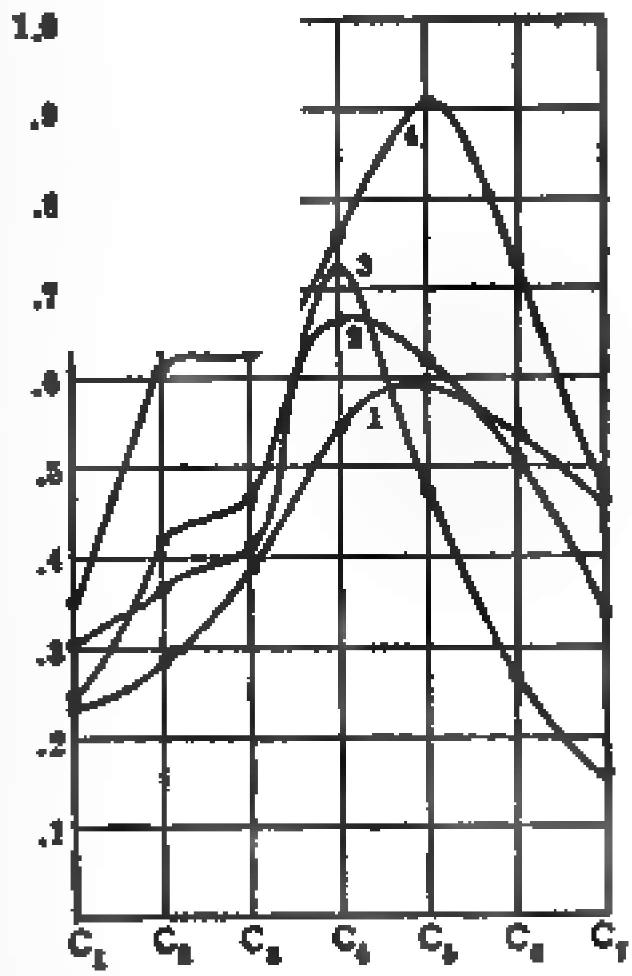


Fig. 7. Absorbing Power of Cushions

is probably the most interesting, because for more standard commercial conditions ordinarily used in churches. This curve is for the elastic-felt cushions of commerce, of elastic cotton covered with ticking and short-nap plush. The absorbing power is per square meter of surface. It is to be observed that all the curves fall off for the higher frequencies, all show a maximum located within an octave, and three of the curves show a curious hump in the second octave. This break in the curve is a genuine phenomenon, as it was tested time after time. It is perhaps due to a SECONDARY RESONANCE, and it is to be observed that it is the more pronounced in those curves that have the sharper resonance in their principal maxima.

Effects of Interference of Sound-Waves. In both articulate speech and music the source of sound is rapidly and, in general, abruptly changing in pitch, quality and loudness. In music one PITCH is held during the length of a note. In articulate speech the unit or ELEMENT OF CONSTANCY is the syllable. Indeed, in speech it is even less than the length of a syllable, for the open-vowel

sound which forms the body of a syllable usually has a consonantal opening and closing. During the constancy of an element, either of music or of speech, a train of sound-waves spreads spherically from the source, just as a train of circular waves spreads outward from a rocking boat on the surface of still water Different portions of this train of spherical waves strike different surfaces of the auditorium and are REFLECTED. After such reflection they begin to cross each other's paths. If their paths are so different in length that one train of waves has entirely passed before the other arrives at a particular point, the only phenomenon at that point is PROLONGATION of the sound. If the space between the two trains of waves is sufficiently great, the effect will be that of an ECHO. If there are a number of such trains of waves thus widely spaced, the effect will be that of MULTIPLE ECHOES. On the other hand, if two trains of waves have traveled so nearly equal paths that they overlap, they will, dependent on the difference in length of the paths which they have traveled, either reinforce or mutually destroy each other. Just as two equal trains of water-waves crossing each other may entirely neutralize each other if the crest of one and the trough of the other arrive together, so two sounds, coming from the same source, in crossing each other may produce silence. This phenomenon is called INTERFERENCE, and is a common phenomenon in all types of wave-motion. Of course, this phenomenon has its complement. If the two trains of water-waves so cross that the crest of one coincides with the crest of the other and trough with trough, the effects will be added together. If the two sound-waves are similarly retarded, the one on the other, their effects will also be added. If the two trains of waves are equal in intensity, the combined intensity will be quadruple that of either of the trains separately, as above explained, or zero, depending on their relative retardation. The effect of this phenomenon is to produce regions in an auditorium of LOUDNESS and regions of comparative or even complete SILENCE. It is a partial explanation of the so-called DEAF REGIONS in an auditorium.

Distribution of Intensity of Sound. It is not difficult to observe this phenomenon directly. It is difficult, however, to measure and record the phenomenon in such a manner as to permit of an accurate chart of the result. Without going into the details of the method employed, the result of these measurements for a room very similar to the Congregational Church in Naugatuck, Conn., is shown in the accompanying chart. The room experimented in was a simple, rectangular room with plain side walls and ends and with a barrel or cylindrical ceiling with the center of curvature at the floor-level. The result is clearly represented in Fig. 8, in which the INTENSITY OF THE SOUND has been indicated by contour-lines in the manner employed in the drawing of the geodetic survey-maps. The phenomenon indicated in these diagrams was not ephemeral, but was constant so long as the source of sound continued, and repeated itself with almost perfect accuracy day after day. Nor was the phenomenon one which could be observed merely instrumentally. To an observer moving about in the room it was quite as striking a phenomenon as the diagram suggests. At the points in the room indicated as HIGH MAXIMA OF INTENSITY in the diagram the sound was so loud as to be disagreeable, at other points so low as to be scarcely audible. It should be added that this distribution of intensity is with the source of sound at the center of the room at the head-level. Had the source of sound been at one end and on the axis of the cylindrical ceiling, the distribution of intensity would still have been bilaterally symmetrical, but not symmetrical about the transverse axis.

Interference-Systems and Reverberation. When a source of sound is maintained constant for a sufficiently long time, a few seconds will ordinarily suffice; the sound becomes steady at every point in the room. The distribution of the

intensity of sound under these conditions is called the INTERFERENCE-SYSTEM, for that particular note, of the room or space in question. If the source of sound is suddenly stopped, it requires some time for the sound in the room to be absorbed. This prolongation of sound after the source has ceased is called REVERBERATION. If the source of sound, instead of being maintained, is short and sharp, it travels as a discrete wave or group of waves about the room, reflected

Fig 8. Distribution of Intensity of Sound

om wall to wall, producing echoes. In the Greek theater there was ordinarily at one echo, "doubling the case-ending," while in the modern auditorium there e many, generally arriving at a less interval of time after the direct sound, and erefore less distinguishable, but stronger and therefore more disturbing.

Photographing Air-Disturbances. The formation and the propagation of HOES may be admirably studied by an adaptation of the so-called SCHLIEREN-

Fig. 9 Photograph of Sound-wave. Vertical Section

Fig. 10. Photograph of Sound-wave. Vertical Section

Fig. 11. Photograph of Sound-wave and Echoes. Vertical Section

Fig. 12. Photograph of Sound-wave and Echoes. Vertical Section

METHODE device for photographing air-disturbances. It is sufficient here to say that the adaptation of this method to the problem in hand consists in the construction of a MODEL in proper scale, of the auditorium to be studied and an inves-

Fig. 13. Photograph of Sound-wave and Echoes. Horizontal Section

tigation of the propagation through it of a proportionally scaled sound-wave. To examine the formation of echoes in a vertical section, the sides of a model are taken off and, as the sound is passing through it, it is illuminated instantaneously

Fig. 14. Photograph of Sound-wave and Echoes. Horizontal Section

by the light from a very fine and somewhat distant electric spark. In the accompanying illustrations, reduced from the photographs, the silhouettes show parts of the shadows cast by the model, and all within are direct photographs of the actual

sound-wave and its echoes. Figs. 9 to 12 show the sound and its echoes at different stages in their propagation through the room, the particular part of the auditorium under investigation being the New Theater in New York City. It

Fig. 15. Photograph of Sound-wave and Echoes. Horizontal Section

is not difficult to identify the MASTER-WAVE and the various ECHOES which it generates, nor, knowing the velocity of sound, to compute the interval at which the echo is heard. To show the generation of echoes and their propagation in

Fig. 16. Photograph of Sound-wave and Echoes. Horizontal Section

a horizontal plane, the ceiling and floor of the model are removed and the photograph taken in a vertical direction. The photographs shown in Figs. 13 to 16 show the echoes produced in the horizontal plane passing through the marble parapet in front of the box.

Solution of Problems Possible in Advance of Construction. While these several factors, REVERBERATION, INTERFERENCE and ECHO, in an auditorium at all complicated are themselves complicated, nevertheless they are capable of an exact solution, or, at least, of a solution as accurate as are the architect's plans in actual construction; and it is entirely possible to calculate in advance of construction whether or not an auditorium will be good, and, if not, to determine the factors contributing to its poor acoustics and a method for its correction.

SPECIFIC GRAVITY

The Specific Gravity of a substance is the number which expresses the ratio that the weight of a given volume of the substance bears to the weight of the same volume of distilled water at a temperature of 62° F.; or, the specific gravity of a body is equal to its weight divided by the weight of an equal volume of water. The specific gravity of a substance, multiplied by the weight of a cubic foot of water, will give the weight of a cubic foot of the given substance. The weight of a cubic foot of water, at 62° F. and at the sea-level, is about 62.355 lb.* The specific gravity of a solid substance may be determined by first weighing a portion of it in air and then in water and dividing the weight in air by the loss of the weight in water; the quotient is the specific gravity required.

Example. A piece of granite weighs 5.32 lb in air; when immersed in water it weighs 3.32 lb.

Solution. Weight in air (5.32 lb) divided by loss of weight in water (2 lb) = 2.66, the specific gravity.

$$2.66 \times 62.355 \text{ lb} = 165.84 \text{ lb} = \text{weight per cubic foot}$$

NOTE. 1 cu ft = 7.48 gal.

* The textbooks differ slightly in regard to this value.

Specific Gravities and Weights per Cubic Foot of Various Substances *

The basis for specific gravities is pure water at 62° F., barometer 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1	Average weight of 1 cu ft lb
Agate, 2.5 to 2.8	2.6	162.1
Air, atmospheric at 60° F., under pressure of one atmosphere, or 14.7 lb per sq in, weight 1/815 the weight of water	0.00123	0.0767
Alabaster, carbonate 2.61 to 2.76	2.68	167.1
Alcohol, absolute, at 32° F	0.794	49.5
Alcohol, 50 per cent	0.934	58.24
Alcohol, 95 per cent	0.815	50.82
Alcohol, commercial	0.833	51.95
Alder, dry † 0.42 to 1.01	0.55	34.3
Alum	0.53	33.0
Aluminum, hammered	2.75	171.7
Aluminum, drawn	2.68	167.1
Aluminum, sheet	2.67	166.5
Aluminum, pure	2.67	166.5
Aluminum, cast	2.56	160.0
Amalgam, 13.7 to 14.1	13.92	868.0
Amber	1.08	67.4
Ambergris	0.87	54.3
Ammonia, 60° F	0.894	55.81
Antimony, cast	6.70	418.0
Antimony, native	6.67	416.0
Apple-wood, dry † 0.66 to 1.25	0.75	46.8
Arsenic 5.7 to 5.8	5.73	357.3
Asbestos 2.1 to 3.0	2.81	175.0
Asbestos sheathing-paper	1.20	75.0
Ash, American white, dry †	0.61	38.0
Ashes of soft coal, solidly packed	0.70	40 to 45
Asphalt, for street-paving	1.60	100.0
Asphaltum 1.11 to 1.23	1.15	69 to 75
Ballast, brick, gravel	1.79	111.6
Bamboo, dry †	0.36	22.5
Barium	3.88	242.0
Barytes	4.45	277.5
Basalt or trap-rock, average	2.96	184.6
Jersey City, N. J	3.00	187.1
Duluth, Minn	2.95	184.0
Staten Island, N. Y	2.86	178.3
Beech, dry † 06.5 to 1.12	0.74	46.0
Beeswax	0.95	59.0
Benzine	0.69	43.0
Beer	1.04	64.9
Birch, dry † 0.52 to 1.08	0.65	40.6
Bismuth, cast 9.76 to 9.90	9.82	612.3
Blood, at 32° F	1.06	66.2
Bone 1.8 to 2.0	1.90	118.6
Borax 1.7 to 1.8	1.75	109.2
Boxwood, French, dry †	1.33	83.0

* The values given in this table are AVERAGE values. In the compilations of these [tab]les the Editor is indebted to Mr. T. Z. Talley for valuable assistance.
See, also, pages 721, 722, and 723.

† The word "dry" in this connection indicates that the wood contains not more than [..]% of moisture. Green timbers usually weigh from one-fifth to nearly one-half more [tha]n dry; ordinary building-timbers, tolerably seasoned, one-sixth more.

Specific Gravities and Weights per Cubic Foot of Various Substances *
(Continued)

The basis for specific gravities is pure water at 62° F., barometer 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1	Average weight of 1 cu ft lb
Boxwood, Dutch, dry † / Boxwood, Brazilian, dry †	1.035	64.5
Brass (copper and zinc), cast .... 7.8 to 9.	8.45	527.0
Brass, rolled	8.56	533.8
Brass, sheet	8.24	513.6
Brass, wire	8.69	542.0
Bricks, common	1.922	120.0
Bricks, light, inferior	1.442	90.0
Bricks, lime-sand	2.163	135.0
Bricks, magnesia	2.643	165.0
Bricks, pressed	2.163	135.0
Bricks, pressed, hard	2.403	150.0
Bricks, soft	1.602	100.0
Bricks, fire / Bricks, paving	2.403	150.0
Brickwork, pressed bricks, fine joints	2.24	140.0
Brickwork, medium quality	2.00	125.0
Brickwork, coarse, inferior, soft	1.60	100.0
Brickwork, at 125 lb per cu ft, 1 cu yd equals 1.507 tons and 17.92 cu ft equal 1 ton	....	....
Bromine	3.19	199.0
Bronze, coin	8.66	540.0
Bronze, gun-metal	8.60	536.3
Bronze, ordinary	8.40	524.0
Bronze, aluminum	7.70	480.0
Butter	0.86	53 to 54
Butternut-tree, dry †	0.38	23.7
Cadmium .... 8.6 to 8.7	8.65	539.4
Calcite .... 2.6 to 2.8	2.70	168.5
Calcium	1.58	98.6
Camphor, dry	0.99	61.7
Caoutchouc (India Rubber)	0.93	58.0
Carbon disulphide	1.29	80.5
Castor-oil	0.96	59.9
Caustic soda	....	88.0
Cedar, red and white, dry †	0.45	28.1
Cement, Natural (Rosendale), loose	1.04	65.0
Cement, Portland, loose	1.35	84.2
Cement, Natural, solid	2.95	183.9
Cement, Portland, solid	3.15	196.6
Chalk	2.35	146.5
Champagne	0.99	61.7
Charcoal of pines and oaks	....	15 to 30
Cherry, dry	0.66	41.2
Chestnut, dry	0.63	39.3
Chromium	5.00	312.0
Cider	1.02	63.5

* The values given in this table for specific gravities and for weights per cubic foot are AVERAGE values.

† The word "dry" in this connection indicates that the wood contains not more than 15% of moisture. Green timbers usually weigh from one-fifth to nearly one-half more than dry; ordinary building-timbers, tolerably seasoned, one-sixth more.

Specific Gravities and Weights per Cubic Foot of Various Substances *
(Continued)

The basis for specific gravities is pure water at 62° F., barometer 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1	Average weight of 1 cu ft lb
Cinnabar	8.12	507.0
Clay, potters', dry ... 1.8 to 2.1	1.90	118.5
Clay, dry, in lump, loose	1.01	63.0
Coal, anthracite, 1.3 to 1.84; of Penn., 1.3 to 1.7	1.50	93.5
Coal, anthracite, broken, of any size, loose, average	....	52 to 56
Coal, anthracite, broken, moderately shaken	....	56 to 60
Coal, anthracite, broken, heaped bushel, loose, 77 to 83 lb	....	....
Coal, anthracite, broken, a ton loose occupies 40 to 43 cu ft	....	....
Coal, bituminous, solid, 1.2 to 1.5	1.35	84.0
Coal, bituminous, solid, Cambria Co., Pa., 1.27 to 1.34	....	79 to 84
Coal, bituminous, broken, of any size, loose	....	47 to 52
Coal, bituminous, moderately shaken	....	51 to 56
Coal, bituminous, a heaped bushel, loose, 70 to 78	....	....
Coal, bituminous, 1 ton occupies 43 to 48 cu ft	....	....
Coke, loose, good quality	....	23 to 32
Coke, loose, a heaped bushel, 35 to 42 lb	....	....
Coke, loose, 1 ton occupies 80 to 97 cu ft	....	....
Concrete, stone ... 130 to 150	2.33	145.0
Concrete, cinder ... 100 to 110	1.68	105.0
Copper, hammered ... 8.8 to 9.0	8.95	558.0
Copper, rolled ... 8.9 to 9.0	8.95	558.0
Copper, drawn wire ... 8.8 to 9.0	8.89	554.5
Copper, sheet	8.72	543.6
Copper, cast ... 8.6 to 8.9	8.82	550.0
Copper, melted	8.23	513.0
Cork, dry	0.24	15.0
Corundum, pure ... 3.92 to 4.01	3.96	247.5
Creosote oil ... 1.04 to 1.10	1.07	66.8
Cypress, American, dry †	0.55	34.3
Dogwood, dry †	0.75	46.8
Douglas fir, dry †	0.51	31.8
Earth, common loam, perfectly dry, loose	....	72 to 80
Earth, common loam, perfectly dry, shaken	....	82 to 92
Earth, common loam, perfectly dry, rammed	....	90 to 100
Earth, common loam, slightly moist, loose	....	70 to 76
Earth, common loam, more moist, loose	....	66 to 68
Earth, common loam, more moist, shaken	....	75 to 90
Earth, common loam, more moist, packed	....	90 to 100
Earth, common loam, as soft, flowing mud	....	104 to 112
Earth, common loam, as soft, flowing mud, well-pressed	....	110 to 120
Ebony	1.22	76.0
Eggs	1.09	68.0
Elder-pith	0.076	4.7
Elm, dry †	0.56	35.0
Elm, rock	0.80	50.0
Emerald	2.70	168.5

* The values given in this table for specific gravities and for weights per cubic foot are AVERAGE values.

† The word "dry" in this connection indicates that the wood contains not more than [illegible] of moisture. Green timbers usually weigh from one-fifth to nearly one-half more than dry; ordinary building-timbers, tolerably seasoned, one-sixth more.

Specific Gravities and Weights per Cubic Foot of Various Substances * (Continued)

The basis for specific gravities is pure water at 62° F., barometer 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1	Average weight of 1 cu ft lb
Emery	4.00	249.5
Fats	0.93	58.0
Feldspar	2.57	160.2
Filbert-tree, dry †	0.60	37.5
Fir, Douglas (see Douglas Fir).		
Flint	2.63	164.0
Gamboge	1.20	74.8
Garnet 3.4 to 4.3	3.85	240.1
Glass, optical	3.45	215.0
Glass, flint	3.00	187.0
Glass, white	2.89	180.2
Glass, plate	2.80	174.6
Glass, green	2.67	166.5
Glass, floor, heavy	2.53	158.0
Glass, window	2.50	156.0
Gneiss (see Granites).		
Gold, pure	19.50	1 215.9
Gold, hammered, native	19.40	1 209.7
Gold, cast	19.258	1 200.8
Granites and gneiss, Connecticut, Greenwich	2.84	177.3
California, Penryn (hornblende)	2.77	172.9
New York	2.74	171.0
Maryland, Port Deposit	2.72	169.6
Massachusetts, Quincy (hornblende)	2.70	168.5
Wisconsin, Athelstane	2.70	168.5
Georgia, Lithornia and Stone Mountain	2.69	167.9
Minnesota	2.68	167.3
California, Rocklin (muscovite)	2.68	167.3
Rhode Island, Westerley	2.67	166.7
Connecticut, New London	2.66	166.0
New Hampshire, Keene	2.66	166.0
Maine, Hallowell	2.65	165.2
New Hampshire, Concord	2.65	165.2
Vermont, Barre	2.65	165.2
Wisconsin, Montello	2.64	164.6
Colorado, Georgetown (biotite)	2.63	164.0
Maine, Fox Island	2.63	164.0
Massachusetts, Rockport	2.61	162.7
Graphite	2.26	140.0
Gravel, dry	1.79	112.0
Gravel, wet	2.00	125.0
Greenstone, trap 2.8 to 3.2	3.00	187.0
Grindstone	2.14	133.5
Gum arabic	1.32	82.5
Gun-metal (see Bronze).		
Gunpowder (granular)	1.00	62.4
Gutta-percha	0.98	61.0

* The values given in this table for specific gravities and for weights per cubic foot are AVERAGE values.

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Specific Gravities and Weights per Cubic Foot of Various Substances *
(Continued)

The basis for specific gravities is pure water at 62° F., barometer 30 in. Weight of 1 cu ft of water 62.355 lb	Average specific gravity. Water = 1	Average weight of 1 cu ft lb
ypsum, natural rock, free from surface-water	2.30	143.0
ypsum, crushed rock, not calcined, all passing 1-in ring	1.52	95.0
ypsum, ground rock, 90% passing 100 mesh, dried not calcined	1.25	78.0
ypsum, Plaster-of-Paris, stucco, stiff mortar, set and dried out	1.22	77.0
ypsum, Plaster-of-Paris, stucco, ground rock, 90% passing 100 mesh, calcined, loose	0.96	60.0
well shaken down or in bins	1.11	70.0
lackmatack (see Larch).	....	....
lay, loose in stacks, about 512 cu ft per ton	....	....
emlock, dry †	0.42	26.2
ickory, pignut, dry †	0.89	55.6
ickory, mocker-nut, dry †	0.85	53.1
ickory, shagbark, dry †	0.81	50.6
ickory, nutmeg, dry †	0.78	48.7
ickory, bitternut, dry †	0.77	48.1
ickory, water, dry †	0.73	45.6
olly	0.76	47.4
oney	1.45	90.5
orn	1.69	105.5
ornblende 3.0 to 3.5	3.25	202.7
e 0.88 to 0.914	0.89	56.0
idiana Limestone	2.31	144.0
dine	4.94	308.0
idium, pure	22.12	1 379.0
on, cast 6.9 to 7.4	7.2	448.9
on, gray, foundry, cold	7.21	450.0
on, gray, foundry, molten	6.94	433.0
on, wrought	7.70	480.0
ory	1.88	117.0
niper-wood	0.57	35.6
aolin	2.20	137.2
va	2.65	165.2
rch, or hackmatack, dry †	0.55	34.3
rd	0.94	58.7
ad, commercial, cast	11.36	708.0
ad, commercial, sheet	11.40	710.8
ad, pure	11.42	713.0
ad, molten	10.40	648.8
gnum-vitæ, dry † 0.65 to 1.33	0.99	41 to 84
me, quick, ground 1.04 to 1.20	1.12	65 to 75
mestone, Illinois	2.57	160.4
Indiana	2.31	144.0
Kentucky	2.685	167.4
Michigan	2.44	152.1
Minnesota	2.655	165.6
Missouri	2.32	144.8
New York	2.71	169.0
Average of limestones	2.57	160.4
nseed-oil	0.935	58.3

The values given in this table for specific gravities and for weights per cubic foot are AGE values.

The word "dry" in this connection indicates that the wood contains not more than of moisture. Green timbers usually weigh from one-fifth to nearly one-half more dry; ordinary building-timbers, tolerably seasoned, one-sixth more.

Specific Gravities and Weights per Cubic Foot of Various Substances*
(Continued)

The basis for specific gravities is pure water at 62° F., barometer 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1	Average weight of 1 cu ft lb
Locust, dry †	0.71	44.3
Magnesite	3.0	187.1
Magnesium, pure	1.72	107.0
Mahogany ....0.56 to 1.06	0.81	50.5
Manganese, pure	8.00	499.0
Manganese, ore, red	4.01	250.0
Manganese, ore, black	3.45	215.1
Marble, average ....2.6 to 164.4	2.6	162.1
domestic,		
New York	2.83	176.5
California	2.75	171.5
Georgia	2.73	170.2
Vermont, Dorset	2.66	166.0
foreign,		
Parian	2.84	177.1
African	2.80	174.6
Carrara	2.72	169.6
Biscayan	2.71	169.0
British	2.71	169.0
French	2.65	165.2
Marl	2.10	131.0
Masonry, brickwork (see Brickwork)		
Masonry, concrete, stone	2.33	145.3
Masonry, concrete, cinder	1.68	105.0
Masonry, granite, dressed	2.64	165.0
Masonry, granite, rubble in cement	2.48	155.0
Masonry, limestone, dressed	2.60	162.0
Masonry, marble, dressed for buildings	2.72	170.0
Masonry, sandstone	2.41	151.0
Mastic, gum resin	0.85	53.0
Mercury, at 32° F	13.62	849.0
Mica ....2.75 to 3.1	2.93	183.0
Milk, at 32° F	1.032	64.3
Molybdenum, pure	8.63	538.1
Mortar, lime	1.65	103.0
Mortar, cement	1.68	105.0
Mud, dry, close	....	80 to 110
Mud, wet, moderately pressed	....	110 to 130
Mud, wet, fluid	....	104 to 120
Mulberry-tree, dry †	0.75	46.8
Naptha-oil, wood, at 32° F	0.85	52.9
Nickel	8.56	517 to 550
Oak, live, dry † ....0.88 to 1.02	0.95	59.3
Oak, white, dry † ....0.66 to 0.88	0.77	48.0
Oak, red and black, dry †	....	32 to 45
Ochre	3.50	218.0
Olive-oil, 32° F	0.916	57.12

* The values given in this table for specific gravities and for weights per cubic foot a[re] AVERAGE values.

† The word "dry" in this connection indicates that the wood contains not more th[an] 15% of moisture. Green timbers usually weigh from one-fifth to nearly one-half m[ore] than dry: ordinary building-timbers, tolerably seasoned, one-sixth more.

Specific Gravities and Weights per Cubic Feet of Various Substances *
(Continued)

The basis for specific gravities is pure water at 62° F., barometer 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1	Average weight of 1 cu ft lb
Oolitic stones	2.25	140.3
Opal	2.15	134.0
Opium	1.34	83.6
Orange-tree	0.71	44.3
Palladium	11.80	735.6
Paper	0.95	59.3
Paraffin	0.88	54.9
Pear-tree wood, dry †	0.67	41.8
Peat, pressed	0.72	45.0
Petroleum, oil	0.878	54.8
Pine, Cuban, dry †	0.63	39.3
Pine, yellow, long-leaf, dry	0.61	38.1
Pine, loblolly, dry	0.53	33.1
Pine, yellow, short-leaf, dry	0.51	31.8
Pine, red, Norway, dry	0.50	31.2
Pine, spruce, dry	0.44	27.5
Pine, white, dry	0.38	23.7
Pitch	1.08	67.0
Plaster of Paris (see Gypsum)	2.25	140.3
Platinum	21.50	1 340.6
Plumbago	2.10	131.0
Poplar, dry †	0.47	29.3
Porcelain, china	2.30	143.4
Porphyry	2.76	172.3
Potash	2.26	141.0
Potassium	0.865	54.0
Pumice-stone	0.92	57.4
Quartz	2.65	165.3
Quince-tree wood, dry †	0.71	44.3
Red lead	8.94	557.5
Resin	1.09	68.0
Rock-crystal	2.60	162.0
Rosewood	0.73	45.6
Rosin	1.10	68.6
Rubber, India	0.93	58.0
Ruby	3.90	243.0
Salt, coarse, per struck bushel, Syracuse, N. Y., 56 lb	....	45.0
Saltpetre	2.02	122 to 130
Sand, of pure quartz, perfectly dry and loose	....	90 to 106
Sand, of pure quartz, voids full of water	....	118 to 129
Sand, of pure quartz, very large and small grains, dry	....	117.0
Sandstone, average	2.44	152.1
Massachusetts, Longmeadow	2.49	155.4
Connecticut, Portland	2.50	156.0
New York .... 2.40 to 2.70	2.60	162.1
New Jersey, Belleville	2.40	149.7
Pennsylvania	2.63	164.2
Virginia, Bristow	2.60	162.0

The values given in this table for specific gravities and for weights per cubic foot ar... RAGE values.

The word "dry" in this connection indicates that the wood contains not more tha... % of moisture. Green timbers usually weigh from one-fifth to nearly one-half mor... n dry; ordinary building-timbers, tolerably seasoned, one-sixth more.

Specific Gravities and Weights per Cubic Foot of Various Substances*
(Continued)

The basis for specific gravities is pure water at 62° F., barometer 30 in. Weight of 1 cu ft of water, 62.355 lb	Average specific gravity. Water = 1	Average weight of 1 cu ft lb
Sandstone, (continued)		
Ohio	2.22	138.6
Michigan	2.35	146.5
Wisconsin	2.22	138.6
Minnesota	2.25	140.5
Colorado	2.33	145.3
California, Angel Island	2.73	170.0
Shales, red or black	2.60	162.0
Silica	2.66	166.0
Silver	10.50	654.5
Slate	2.81	175.0
Snow, freshly fallen	....	5 to 12
Snow, moistened, compacted by rain	....	15 to 50
Soapstone	2.73	170.0
Sodium	0.978	61.0
Spelter	7.10	443 c
Spirit, rectified	0.824	51 4
Spruce	0.40	25 0
Steel, cast	7.9	492 6
Steel, wrought	7.85	489.6
Sugar	1.60	100.0
Sycamore, dry	0.58	36.5
Talc	2.81	175.2
Tallow	0.94	58.6
Tamarack	0.38	23.6
Tar	1.00	62.4
Teak	0.70	43.7
Terra-cotta, solid blocks	....	120 to 122
hollow blocks, 1½-in thick, smaller pieces heaviest	....	65 to 85
Tiles, solid	2.20	136.5
Tin, rolled	7.40	461.5
Tin, cast	7.30	455.0
Tin, molten	7.02	437.7
Trap (see Basalt).		
Tungsten	19.129	1 192.8
Turpentine	0.87	54.3
Type-metal, cast	10.45	651.8
Uranium	18.49	1 153.0
Vinegar	1.08	67.5
Walnut, black, dry	0.60	37.5
Water, pure rain, distilled, at 32° F., barometer 30 in.	....	62.4
Water, pure rain, distilled, at 62° F., barometer 30 in.	1.00	62.355
Water, pure rain, distilled, at 212° F., barometer 30 in.	....	59.7
Water, sea .... 1.026 to 1.030	1.028	64.1
Wax (see Beeswax).		
Willow	0.49	30.5
Wine	1.01	63.0
Zinc or spelter .... 6.8 to 7.2	7.00	436.5

* The values given in this table for specific gravities and for weights per cubic foot are AVERAGE values.

† The word "dry" in this connection indicates that the wood contains not more than 15% of moisture. Green timbers usually weigh from one-fifth to nearly one-half more than dry; ordinary building-timbers, tolerably seasoned, one-sixth more.

WIRE-GAUGES* AND METAL-DATA

A **Wire-Gauge** is a method of designating the diameter of wires or the thick-
ss of sheets of metal by the numbers of a table arranged on a certain fixed basis.
ere are at the present time several gauges, resulting in great confusion,
ble XIII, page 402, gives the diameters of the gauges in common use. The
y legal gauge in this country is the United States standard gauge, described
page 1600. It is used by most of the manufacturers of sheet iron and steel
l tin plate. The Brown & Sharpe gauge is commonly used for designating
: of copper wires (see page 1510); also for sheet copper and brass. Nearly
copper wire, bare and insulated, is ordered, manufactured, and carried in
ck in accordance with this gauge. This might be called the Copper Wire
uge. The American Steel & Wire Company uses the old Washburn &
en and Roebling gauges for all their steel and iron wire and also for wire nails.
: sectional areas for these gauges are given on pages 403 and 1512, taken from
Roebling and American Steel & Wire Company's lists. When placing orders
sheets and wire, it is always best to specify the weight per square or linear
: or the thickness or diameter in thousandths of an inch or in circular mils.
: gauge for steel wire, used by the J. A. Roebling's Sons Company, is given
page 403, and the circular-mil gauge on page 1473. The gauge used by this
ipany is the same as the Washburn & Moen gauge, or the American Steel
Vire gauge, except that the diameters in most cases are given to the nearest
This gauge is so generally used for steel wire that it is sometimes called
Steel Wire Gauge or the Market Wire Gauge. The Birmingham Wire gauge
ie same as Stubs' Iron-Wire gauge, but entirely different from Stubs' Steel-
e gauge. Galvanized telegraph and telephone-wire, both bare and insu-
d, and galvanized armor-wire are usually designated by this gauge. Its
is not very extensive and is becoming less. The new British Standard gauge
he legal standard for Great Britain and is used there for all kinds of wire.
use in this country is very limited. It is known, also, as the English Legal
idard gauge and the Imperial Wire gauge.

* See, also, pages 401, 402, 403, 1469, 1473, 1510, 1512, and 1600.

Weights in Pounds per Square Foot of Sheets of Wrought Iron, Steel, Copper, and Brass

Thickness by American (Brown & Sharpe) gauge *

No. of gauge	Thickness in inches	Iron	Steel	Copper	Brass
0000	0.46	18.40	18.77	20.84	19.69
000	0.4096	16.39	16.71	18.56	17.53
00	0.3648	14.59	14.88	16.53	15.61
0	0.3249	12.99	13.25	14.72	13.90
1	0.2893	11.57	11.80	13.11	12.38
2	0.2576	10.31	10.51	11.67	11.03
3	0.2294	9.18	9.36	10.39	9.82
4	0.2043	8.17	8.34	9.26	8.74
5	0.1819	7.28	7.42	8.24	7.79
6	0.1620	6.48	6.61	7.34	6.93
7	0.1443	5.77	5.89	6.54	6.18
8	0.1285	5.14	5.24	5.82	5.50
9	0.1144	4.58	4.67	5.18	4.90
10	0.1019	4.08	4.16	4.62	4.36
11	0.0907	3.63	3.70	4.11	3.88
12	0.0808	3.23	3.30	3.66	3.46
13	0.0720	2.88	2.94	3.26	3.08
14	0.0641	2.56	2.61	2.90	2.74
15	0.0571	2.28	2.33	2.59	2.44
16	0.0508	2.03	2.07	2.30	2.18
17	0.0453	1.81	1.85	2.05	1.94
18	0.0403	1.61	1.64	1.83	1.73
19	0.0359	1.44	1.46	1.63	1.54
20	0.0320	1.28	1.30	1.45	1.37
21	0.0285	1.14	1.16	1.29	1.22
22	0.0253	1.01	1.03	1.15	1.08
23	0.0226	0.903	0.921	1.02	0.966
24	0.0201	0.804	0.820	0.911	0.860
25	0.0179	0.716	0.730	0.811	0.766
26	0.0159	0.638	0.650	0.722	0.682
27	0.0142	0.568	0.579	0.643	0.608
28	0.0126	0.506	0.516	0.573	0.541
29	0.0113	0.450	0.459	0.510	0.482
30	0.0100	0.401	0.409	0.454	0.429
31	0.0089	0.357	0.364	0.404	0.382
32	0.0080	c.318	0.324	0.360	0.340
33	0.0071	0.283	0.289	0.321	0.303
34	0.0063	0.252	0.257	0.286	0.270
35	0.0056	0.224	0.229	0.254	0.240
Specific gravity		7.704	7.85	8.72	8.24
Weight per cubic foot		480.00	489.60	543.6	513.6
Weight per cubic inch		0.2778	0.2833	0.3146	0.2972

* For other gauges see pages 401, 402, 403, 1469, 1473, 1509, 1512, and 1600.

Thickness or diameter, in	Lead: Sheets, per sq ft, lb	Lead: Square bars 1 ft long, lb	Lead: Round bars 1 ft long, lb	Copper: Sheets, per sq ft, lb	Copper: Square bars 1 ft long, lb	Copper: Round bars 1 ft long, lb	Brass: Sheets, per sq ft, lb	Brass: Square bars 1 ft long, lb	Brass: Round bars 1 ft long, lb	Thickness or diameter, in
1/32	1.86	0.005	0.004	1.44	0.004	0.003	1.36	0.004	0.003	1/32
1/16	3.72	0.019	0.015	2.89	0.015	0.012	2.71	0.014	0.011	1/16
3/32	5.58	0.044	0.034	4.33	0.034	0.027	4.06	0.032	0.025	3/32
1/8	7.44	0.078	0.061	5.77	0.060	0.047	5.42	0.056	0.044	1/8
5/32	9.30	0.121	0.095	7.20	0.094	0.074	6.75	0.088	0.069	5/32
3/16	11.20	0.174	0.137	8.66	0.135	0.106	8.13	0.127	0.100	3/16
7/32	13.00	0.237	0.187	10.10	0.184	0.144	9.50	0.173	0.136	7/32
1/4	14.90	0.310	0.244	11.50	0.240	0.189	10.80	0.226	0.177	1/4
5/16	18.60	0.485	0.381	14.40	0.376	0.295	13.50	0.353	0.277	5/16
3/8	22.30	0.698	0.548	17.30	0.541	0.425	16.30	0.508	0.399	3/8
7/16	26.00	0.950	0.746	20.30	0.736	0.578	19.00	0.691	0.543	7/16
1/2	29.80	1.240	0.974	23.10	0.962	0.755	21.70	0.903	0.709	1/2
9/16	33.50	1.570	1.230	26.00	1.220	0.955	24.30	1.140	0.900	9/16
5/8	37.20	1.940	1.520	28.90	1.500	1.180	27.10	1.410	1.110	5/8
11/16	40.90	2.340	1.840	31.70	1.820	1.430	29.80	1.700	1.340	11/16
3/4	44.60	2.790	2.190	34.60	2.160	1.700	32.50	2.030	1.600	3/4
13/16	48.30	3.270	2.570	37.50	2.550	1.990	35.20	2.380	1.870	13/16
7/8	52.10	3.800	2.980	40.40	2.940	2.310	37.90	2.760	2.170	7/8
15/16	56.00	4.370	3.420	43.30	3.380	2.650	40.60	3.180	2.490	15/16
1	59.50	4.960	3.900	46.20	3.850	3.020	43.30	3.610	2.840	1
1 1/8	66.90	6.270	4.920	52.00	4.870	3.820	48.70	4.570	3.600	1 1/8
1 1/4	74.40	7.750	6.090	57.70	6.010	4.720	54.20	5.640	4.430	1 1/4
1 3/8	81.80	9.370	7.370	63.50	7.280	5.720	59.60	6.820	5.370	1 3/8
1 1/2	89.30	11.200	8.770	69.30	8.650	6.800	65.00	8.120	6.380	1 1/2
1 5/8	96.70	13.100	10.30	75.10	10.200	7.980	70.40	9.530	7.490	1 5/8
1 3/4	104.00	15.200	11.90	80.80	11.800	9.250	75.90	11.100	8.680	1 3/4
1 7/8	112.00	17.500	13.70	86.60	13.500	10.600	81.30	12.700	9.970	1 7/8
2	119.00	19.800	15.60	92.30	15.400	12.100	86.70	14.400	11.300	2

Sizes and Weights of Smooth Steel Wire *

As made by the American Steel & Wire Company

No. of gauge	Diameters: Fractions of inch	Diameters: Decimals of inch	Diameters: Milli-meters	Sectional area, sq in	Weight †: Pounds per 100 feet	Weight †: Pounds per mile	No. of feet per pound
000000	....	0.4615	11.72	0.16728	56.81	2999.0	1.76
......	7/16	0.4375	11.11	0.15033	51.05	2696.0	1.959
00000	....	0.4305	10.93	0.14556	49.43	2610.0	2.023
......	13/32	0.40625	10.32	0.12962	44.02	2324.0	2.272
0000	....	0.3938	10.00	0.12180	41.36	2184.0	2.418
......	3/8	0.3750	9.525	0.11045	37.51	1980.0	2.666
000	....	0.3625	9.2075	0.10321	35.05	1851.0	2.853
......	11/32	0.34375	8.731	0.092806	31.52	1664.0	3.173
00	....	0.3310	8.407	0.086049	29.22	1543.0	3.422
......	5/16	0.3125	7.938	0.076699	26.05	1375.0	3.839
0	....	0.3065	7.785	0.073782	25.06	1323.0	3.991
1	....	0.2830	7.188	0.062902	21.36	1128.0	4.681
......	9/32	0.28125	7.144	0.062126	21.10	1114.0	4.740
2	....	0.2625	6.668	0.054119	18.38	970.4	5.441
......	1/4	0.2500	6.350	0.049087	16.67	880.2	5.999
3	....	0.2437	6.190	0.046645	15.84	836.4	6.313
4	....	0.2253	5.723	0.039867	13.54	714.8	7.386
......	7/32	0.21875	5.556	0.037583	12.76	673.9	7.835
5	....	0.2070	5.258	0.033654	11.43	603.4	8.750
6	....	0.1920	4.877	0.028953	9.832	519.2	10.17
......	3/16	0.1875	4.763	0.027612	9.377	495.1	10.66
7	....	0.1770	4.496	0.024606	8.356	441.2	11.97
8	....	0.1620	4.115	0.020612	7.000	369.6	14.29
......	5/32	0.15625	3.969	0.019175	6.512	343.8	15.36
9	....	0.1483	3.767	0.017273	5.866	309.7	17.05
10	....	0.1350	3.429	0.014314	4.861	256.7	20.57
......	1/8	0.125	3.175	0.012272	4.168	220.0	24.00
11	....	0.1205	3.061	0.011404	3.873	204.5	25.82
12	....	0.1055	2.680	0.0087417	2.969	156.7	33.69
......	3/32	0.09375	2.381	0.0069029	2.344	123.8	42.66
13	....	0.0915	2.324	0.0065755	2.233	117.9	44.78
14	....	0.0800	2.032	0.0050266	1.707	90.13	58.58
15	....	0.0720	1.829	0.0040715	1.383	73.01	72.32
16	1/16	0.0625	1.588	0.0030680	1.042	55.01	95.98
17	....	0.0540	1.372	0.0022902	0.7778	41.07	128.60
18	....	0.0475	1.207	0.0017721	0.6018	31.77	166.20
19	....	0.0410	1.041	0.0013203	0.4484	23.67	223.00
20	....	0.0348	0.8839	0.00095115	0.3230	17.05	309.60
21	....	0.0317	0.8052	0.00078924	0.2680	14.15	373.10
......	1/32	0.03125	0.7938	0.00076699	0.2605	13.75	383.00
22	....	0.0286	0.7264	0.00064242	0.2182	11.52	458.40
23	....	0.0258	0.6553	0.00052279	0.1775	9.37	563.30
24	....	0.0230	0.5842	0.00041548	0.1411	7.45	708.70

* For other gauges, see pages 401, 402, 403, 1469, 1473, 1509, 1510 and 1600.

† For iron wire, the values in columns 6 and 7 should be multiplied by 0.98 and [illegible] copper wire, by 1.12.

Kinds of Wire Manufactured by the American Steel and Wire Company

Market-wire, Nos. 0000 to 18.

Annealed stone-wire or weaving-wire, Nos. 16 to 47.

Tinned market-wire, Nos. 0 to 18.

Tinned stone-wire, Nos. 16 to 40.

Gun-screw wire, finished with great care as regards roundness and exactness o gauge, Nos. 18 to 50.

Machinery-wire, Nos. 00000 to 18.

Cast-steel wire, ½-in diameter, down to No. 26.

Drill and needle-steel wire, Nos. 12 to 25.

The term MARKET-WIRE applies to the ordinary and most used forms of Besemer ANNEALED, BRIGHT, GALVANIZED, TINNED and COPPERED wires.

Galvanized-Iron-Wire Strand. The diameter, list-price per 100 ft, weight er 100 feet and approximate breaking-load in pounds for this wire is given in 'able XVI, Chapter XI.

Weights and Areas of Square and Round Bars and Circumferences of Round Steel Bars*

Weights are for steel, at 489.6 lb per cu ft

Thickness or diameter, in	Weight of □ bar 1 ft long, lb	Weight of ○ bar 1 ft long, lb	Area of □ bar, sq in	Area of ○ bar, sq in	Circumference of ○ bar, in
1/16	0.013	0.010	0.0039	0.0031	0.1963
5/64	0.021	0.016	0.0061	0.0048	0.2454
3/32	0.030	0.023	0.0088	0.0069	0.2945
7/64	0.041	0.032	0.0120	0.0094	0.3436
1/8	0.053	0.042	0.0156	0.0123	0.3927
9/64	0.067	0.053	0.0198	0.0155	0.4418
5/32	0.083	0.065	0.0244	0.0192	0.4909
11/64	0.100	0.079	0.0295	0.0232	0.5400
3/16	0.120	0.094	0.0352	0.0276	0.5890
13/64	0.140	0.110	0.0413	0.0324	0.6381
7/32	0.163	0.128	0.0479	0.0376	0.6872
15/64	0.187	0.147	0.0549	0.0431	0.7363
1/4	0.213	0.167	0.0625	0.0491	0.7854
17/64	0.240	0.188	0.0706	0.0554	0.8345
9/32	0.269	0.211	0.0791	0.0621	0.8836
19/64	0.300	0.235	0.0881	0.0692	0.9327
5/16	0.332	0.261	0.0977	0.0767	0.9817
11/32	0.402	0.316	0.1182	0.0928	1.0799
3/8	0.478	0.376	0.1406	0.1104	1.1781
13/32	0.561	0.441	0.1650	0.1296	1.2763
7/16	0.651	0.511	0.1914	0.1503	1.3744
15/32	0.747	0.587	0.2197	0.1726	1.4726
1/2	0.850	0.668	0.2500	0.1963	1.5708
17/32	0.960	0.754	0.2822	0.2217	1.6690
9/16	1.076	0.845	0.3164	0.2485	1.7671
19/32	1.199	0.941	0.3525	0.2769	1.8653
5/8	1.328	1.043	0.3906	0.3068	1.9635
11/16	1.607	1.262	0.4727	0.3712	2.1598
3/4	1.913	1.502	0.5625	0.4418	2.3562
13/16	2.245	1.763	0.6602	0.5185	2.5525
7/8	2.603	2.044	0.7656	0.6013	2.7489
15/16	2.989	2.347	0.8789	0.6903	2.9452

* Adapted from the 1919 Edition of the Handbook of the Cambria Steel Company, Johnstown, Pa.

Weights and Areas of Square and Round Steel Bars *

Weights are for steel, at 489.6 lb per cu ft

Thickness, in	□ Area, sq in	□ Weight per foot, lb	○ Area, sq in	○ Weight per foot, lb	Thickness, in	□ Area, sq in	□ Weight per foot, lb	○ Area, sq in	○ Weight per foot, lb
1	1.000	3.400	0.785	2.670	3	9.000	30.60	7.069	24.03
1/16	1.129	3.838	0.887	3.014	1/16	9.379	31.89	7.366	25.04
1/8	1.266	4.303	0.994	3.379	1/8	9.766	33.20	7.670	26.08
3/16	1.410	4.795	1.108	3.766	3/16	10.16	34.55	7.980	27.13
1/4	1.563	5.312	1.227	4.173	1/4	10.56	35.92	8.296	28.20
5/16	1.723	5.857	1.353	4.600	5/16	10.97	37.31	8.618	29.30
3/8	1.891	6.428	1.485	5.049	3/8	11.39	38.73	8.946	30.42
7/16	2.066	7.026	1.623	5.518	7/16	11.82	40.18	9.281	31.56
1/2	2.250	7.650	1.767	6.008	1/2	12.25	41.65	9.621	32.71
9/16	2.441	8.301	1.918	6.520	9/16	12.69	43.14	9.968	33.90
5/8	2.641	8.978	2.074	7.051	5/8	13.14	44.68	10.32	35.09
11/16	2.848	9.682	2.237	7.604	11/16	13.60	46.24	10.68	36.31
3/4	3.063	10.41	2.405	8.178	3/4	14.06	47.82	11.05	37.56
13/16	3.285	11.17	2.580	8.773	13/16	14.54	49.42	11.42	38.81
7/8	3.516	11.95	2.761	9.388	7/8	15.02	51.05	11.79	40.10
15/16	3.754	12.76	2.948	10.02	15/16	15.50	52.71	12.18	41.40
2	4.000	13.60	3.142	10.68	4	16.00	54.40	12.57	42.73
1/16	4.254	14.46	3.341	11.36	1/16	16.50	56.11	12.96	44.07
1/8	4.516	15.35	3.547	12.06	1/8	17.02	57.85	13.36	45.44
3/16	4.785	16.27	3.758	12.78	3/16	17.54	59.62	13.77	46.83
1/4	5.063	17.22	3.976	13.52	1/4	18.06	61.41	14.19	48.24
5/16	5.348	18.19	4.200	14.28	5/16	18.60	63.23	14.61	49.66
3/8	5.641	19.18	4.430	15.07	3/8	19.14	65.08	15.03	51.11
7/16	5.941	20.20	4.666	15.86	7/16	19.69	66.95	15.47	52.58
1/2	6.250	21.25	4.909	16.69	1/2	20.25	68.85	15.90	54.07
9/16	6.566	22.33	5.157	17.53	9/16	20.82	70.78	16.35	55.59
5/8	6.891	23.43	5.412	18.40	5/8	21.39	72.73	16.80	57.12
11/16	7.223	24.56	5.673	19.29	11/16	21.97	74.70	17.26	58.67
3/4	7.563	25.71	5.940	20.20	3/4	22.56	76.71	17.72	60.25
13/16	7.910	26.90	6.213	21.12	13/16	23.16	78.74	18.19	61.84
7/8	8.266	28.10	6.492	22.07	7/8	23.77	80.81	18.67	63.46
15/16	8.629	29.34	6.777	23.04	15/16	24.38	82.89	19.15	65.10

* Adapted from the 1919 Edition of the Handbook of the Cambria Steel Company
nstown, Pa.

Weights and Areas of Square and Round Steel Bars * (Continued)

Weights are for steel, at 489.6 lb per cu ft

Thickness, in	□ Area, sq in	□ Weight per foot, lb	○ Area, sq in	○ Weight per foot, lb	Thickness, in	□ Area, sq in	□ Weight per foot, lb	○ Area, sq in	○ Weight per foot, lb
5	25.00	85.00	19.64	66.76	7	49.00	166.6	38.49	130.9
1/16	25.63	87.14	20.13	68.44	1/4	52.56	178.7	41.28	140.4
1/8	26.27	89.30	20.63	70.14	1/2	56.25	191.3	44.18	150.2
3/16	26.91	91.49	21.14	71.86	3/4	60.06	204.2	47.17	160.3
1/4	27.56	93.72	21.65	73.60	8	64.00	217.6	50.27	171.0
5/16	28.22	95.96	22.17	75.37	1/4	68.06	231.4	53.46	181.8
3/8	28.89	98.23	22.69	77.15	1/2	72.25	245.6	56.75	193.0
7/16	29.57	100.5	23.22	78.95	3/4	76.56	260.3	60.13	204.4
1/2	30.25	102.8	23.76	80.77	9	81.00	275.4	63.62	216.3
9/16	30.94	105.2	24.30	82.62	1/4	85.56	290.9	67.20	228.5
5/8	31.64	107.6	24.85	84.49	1/2	90.25	306.8	70.88	241.0
11/16	32.35	110.0	25.41	86.38	3/4	95.06	323.2	74.66	253.9
3/4	33.06	112.4	25.97	88.29	10	100.0	340.0	78.54	267.0
13/16	33.79	114.9	26.54	90.22	1/4	105.1	357.2	82.52	280.6
7/8	34.52	117.4	27.11	92.17	1/2	110.3	374.9	86.59	294.4
15/16	35.25	119.9	27.69	94.14	3/4	115.6	392.9	90.76	308.6
6	36.00	122.4	28.27	96.14	11	121.0	411.4	95.03	323.1
1/8	37.52	127.6	29.47	100.2	1/4	126.6	430.3	99.40	337.9
1/4	39.06	132.8	30.68	104.3	1/2	132.3	449.6	103.9	353.1
3/8	40.64	138.2	31.92	108.5	3/4	138.1	469.4	108.4	368.6
1/2	42.25	143.6	33.18	112.8	12	144.0	489.6	113.1	384.5
5/8	43.89	149.2	34.47	117.2	.....	.....	......	......	......
3/4	45.56	154.9	35.79	121.7	.....	.....	......	......	......
7/8	47.27	160.8	37.12	126.2	.....	.....	......	......	......

* Adapted from the 1919 Edition of the Handbook of the Cambria Steel Company, Johnstown, Pa.

Weights in Pounds of Flat Rolled Steel Bars

PER LINEAR FOOT

One cubic foot of steel weighs 489.6 lb

For thicknesses from 1/16 in to 9/16 in and widths from 1/4 in to 3/4 in

Thickness, inches	Width of bar, inches								
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4
1/16	0.053	0.066	0.080	0.093	0.106	0.120	0.133	0.146	0.159
5/64	0.066	0.083	0.100	0.116	0.133	0.149	0.166	0.183	0.199
3/32	0.080	0.100	0.120	0.139	0.159	0.179	0.199	0.219	0.239
7/64	0.093	0.116	0.139	0.163	0.186	0.209	0.232	0.256	0.279
1/8	0.106	0.133	0.159	0.186	0.212	0.239	0.266	0.292	0.319
9/64	0.120	0.149	0.179	0.209	0.239	0.269	0.299	0.329	0.359
5/32	0.133	0.166	0.199	0.232	0.266	0.299	0.332	0.365	0.398
11/64	0.146	0.183	0.219	0.256	0.292	0.329	0.365	0.402	0.438
3/16	0.159	0.199	0.239	0.279	0.319	0.359	0.398	0.438	0.478
13/64	0.173	0.216	0.259	0.302	0.345	0.388	0.432	0.475	0.518
7/32	0.186	0.232	0.279	0.325	0.372	0.418	0.465	0.511	0.558
15/64	0.199	0.249	0.299	0.349	0.398	0.448	0.498	0.548	0.598
1/4	0.213	0.266	0.319	0.372	0.425	0.478	0.531	0.584	0.638
17/64	0.226	0.282	0.339	0.395	0.452	0.508	0.564	0.621	0.677
9/32	0.239	0.299	0.359	0.418	0.478	0.538	0.598	0.657	0.717
19/64	0.252	0.315	0.379	0.442	0.505	0.568	0.631	0.694	0.757
5/16	0.266	0.332	0.398	0.465	0.531	0.598	0.664	0.730	0.797
21/64	0.279	0.349	0.418	0.488	0.558	0.628	0.697	0.767	0.827
11/32	0.292	0.365	0.438	0.511	0.584	0.657	0.730	0.804	0.877
23/64	0.305	0.382	0.458	0.535	0.611	0.687	0.764	0.840	0.916
3/8	0.319	0.398	0.478	0.558	0.638	0.717	0.797	0.877	0.956
25/64	0.332	0.415	0.498	0.581	0.664	0.747	0.830	0.913	0.996
13/32	0 345	0.432	0.518	0.604	0.691	0.777	0.863	0.950	1.04
27/64	0.359	0.448	0.538	0.628	0.717	0.807	0.896	0.986	1.08
7/16	0.372	0.465	0.558	0.651	0.744	0.837	0.930	1.02	1.12
29/64	0.385	0.481	0.578	0.674	0.770	0.867	0.963	1.06	1.16
15/32	0.398	0.498	0.598	0.697	0.797	0.896	0.996	1.10	1.20
31/64	0.412	0.515	0.618	0.721	0.823	0.926	1.03	1.13	1.24
1/2	0.425	0.531	0.638	0.744	0.850	0.956	1.06	1.17	1.28
33/64	0.438	0.548	0.657	0.767	0.877	0.986	1.10	1.21	1.31
17/32	0.452	0.564	0.677	0.790	0.903	1.02	1.13	1.24	1.35
35/64	0.465	0.581	0.697	0.813	0.930	1.05	1.16	1.28	1.39
9/16	0.478	0.598	0.717	0.837	0.956	1.08	1.20	1.31	1.43

Weights in Pounds of Flat Rolled Steel Bars (Continued)

PER LINEAR FOOT

For thicknesses from 1/16 to 2 in and widths from 1 to 3 in

Thickness, inches	Width of bar, inches: 1	1¼	1½	1¾	2	2¼	2½	2¾	3
1/16	0.21	0.26	0.32	0.37	0.43	0.48	0.53	0.58	0.63
⅛	0.42	0.53	0.64	0.75	0.85	0.96	1.06	1.17	1.28
3/16	0.63	0.79	0.96	1.11	1.28	1.44	1.59	1.75	1.91
¼	0.85	1.06	1.28	1.49	1.70	1.91	2.12	2.34	2.55
5/16	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2.92	3.19
⅜	1.28	1.59	1.92	2.23	2.55	2.87	3.19	3.51	3.83
7/16	1.49	1.86	2.23	2.60	2.98	3.35	3.72	4.09	4.46
½	1.70	2.12	2.55	2.98	3.40	3.83	4.25	4.67	5.10
9/16	1.92	2.39	2.87	3.35	3.83	4.30	4.78	5.26	5.74
⅝	2.12	2.65	3.19	3.72	4.25	4.78	5.31	5.84	6.38
11/16	2.34	2.92	3.51	4.09	4.67	5.26	5.84	6.43	7.02
¾	2.55	3.19	3.83	4.47	5.10	5.75	6.38	7.02	7.65
13/16	2.76	3.45	4.14	4.84	5.53	6.21	6.90	7.60	8.29
⅞	2.98	3.72	4.47	5.20	5.95	6.69	7.44	8.18	8.93
15/16	3.19	3.99	4.78	5.58	6.38	7.18	7.97	8.77	9.57
1	3.40	4.25	5.10	5.95	6.80	7.65	8.50	9.35	10.20
1 1/16	3.61	4.52	5.42	6.32	7.22	8.13	9.03	9.93	10.84
1⅛	3.83	4.78	5.74	6.70	7.65	8.61	9.57	10.52	11.48
1 3/16	4.04	5.05	6.06	7.07	8.08	9.09	10.10	11.11	12.12
1¼	4.25	5.31	6.38	7.44	8.50	9.57	10.63	11.69	12.75
1 5/16	4.46	5.58	6.69	7.81	8.93	10.04	11.16	12.27	13.39
1⅜	4.67	5.84	7.02	8.18	9.35	10.52	11.69	12.85	14.03
1 7/16	4.89	6.11	7.34	8.56	9.78	11.00	12.22	13.44	14.66
1½	5.10	6.38	7.65	8.93	10.20	11.48	12.75	14.03	15.30
1 9/16	5.32	6.64	7.97	9.30	10.63	11.95	13.28	14.61	15.94
1⅝	5.52	6.90	8.29	9.67	11.05	12.43	13.81	15.19	16.58
1 11/16	5.74	7.17	8.61	10.04	11.47	12.91	14.34	15.78	17.22
1¾	5.95	7.44	8.93	10.42	11.90	13.40	14.88	16.37	17.85
1 13/16	6.16	7.70	9.24	10.79	12.33	13.86	15.40	16.95	18.49
1⅞	6.38	7.97	9.57	11.15	12.75	14.34	15.94	17.53	19.13
1 15/16	6.59	8.24	9.88	11.53	13.18	14.83	16.47	18.12	19.77
2	6.80	8.50	10.20	11.90	13.60	15.30	17.00	18.70	20.40

Weights in Pounds of Flat Rolled Steel Bars (Continued)

PER LINEAR FOOT

For thicknesses from 1/16 to 2 in and widths from 3½ to 7½ in

Thickness, inches	Width of bar, inches								
	3½	4	4½	5	5½	6	6½	7	7½
1/16	0.75	0.85	0.96	1.06	1.17	1.28	1.39	1.49	1.60
⅛	1.49	1.70	1.92	2.13	2.34	2.55	2.77	2.98	3.19
3/16	2.23	2.55	2.87	3.19	3.51	3.83	4.14	4.46	4.78
¼	2.98	3.40	3.83	4.25	4.67	5.10	5.53	5.95	6.36
5/16	3.72	4.25	4.78	5.31	5.84	6.38	6.90	7.44	7.97
⅜	4.47	5.10	5.74	6.38	7.02	7.65	8.29	8.93	9.57
7/16	5.20	5.95	6.70	7.44	8.18	8.93	9.67	10.41	11.16
½	5.95	6.80	7.65	8.50	9.35	10.20	11.05	11.90	12.75
9/16	6.70	7.65	8.61	9.57	10.52	11.48	12.43	13.39	14.34
⅝	7.44	8.50	9.57	10.63	11.69	12.75	13.81	14.87	15.94
11/16	8.18	9.35	10.52	11.69	12.85	14.03	15.20	16.36	17.53
¾	8.93	10.20	11.48	12.75	14.03	15.30	16.58	17.85	19.13
13/16	9.67	11.05	12.43	13.81	15.19	16.58	17.95	19.34	20.72
⅞	10.41	11.90	13.39	14.87	16.36	17.85	19.34	20.83	22.32
15/16	11.16	12.75	14.34	15.94	17.53	19.13	20.72	22.32	23.91
1	11.90	13.60	15.30	17.00	18.70	20.40	22.10	23.80	25.50
1 1/16	12.65	14.45	16.26	18.06	19.87	21.68	23.48	25.29	27.10
1⅛	13.39	15.30	17.22	19.13	21.04	22.95	24.87	26.78	28.68
1 3/16	14.13	16.15	18.17	20.19	22.21	24.23	26.24	28.26	30.28
1¼	14.87	17.00	19.13	21.25	23.38	25.50	27.62	29.75	31.88
1 5/16	15.62	17.85	20.08	22.32	24.54	26.78	29.01	31.23	33.48
1⅜	16.36	18.70	21.04	23.38	25.71	28.05	30.39	32.72	35.06
1 7/16	17.10	19.85	21.99	24.44	26.88	29.33	31.77	34.21	36.66
1½	17.85	20.40	22.95	25.50	28.05	30.60	33.15	35.70	38.26
1 9/16	18.60	21.25	23.91	26.57	29.22	31.88	34.53	37.19	39.84
1⅝	19.34	22.10	24.87	27.63	30.39	33.15	35.91	38.67	41.44
1 11/16	20.08	22.95	25.82	28.69	31.55	34.43	37.30	40.16	43.03
1¾	20.83	23.80	26.78	29.75	32.73	35.70	38.68	41.65	44.63
1 13/16	21.57	24.65	27.73	30.81	33.89	36.98	40.05	43.14	46.22
1⅞	22.31	25.50	28.69	31.87	35.06	38.25	41.44	44.63	47.82
1 15/16	23.06	26.35	29.64	32.94	36.23	39.53	42.82	46.12	49.41
2	23.80	27.20	30.60	34.00	37.40	40.80	44.20	47.60	51.00

Weights in Pounds of Flat Rolled Steel Bars (Continued)

PER LINEAR FOOT

For thicknesses from 1/16 to 2 in and widths from 8 to 12 in

Thickness, inches	Width of bar, inches								
	8	8½	9	9½	10	10½	11	11½	12
1/16	1.70	1.81	1.91	2.02	2.13	2.23	2.34	2.45	2.55
1/8	3.40	3.61	3.82	4.04	4.25	4.46	4.68	4.89	5.10
3/16	5.10	5.42	5.74	6.06	6.38	6.70	7.02	7.32	7.65
1/4	6.80	7.22	7.65	8.08	8.50	8.92	9.34	9.78	10.20
5/16	8.50	9.03	9.56	10.10	10.62	11.16	11.68	12.22	12.75
3/8	10.20	10.84	11.48	12.12	12.75	13.39	14.03	14.68	15.30
7/16	11.90	12.64	13.40	14.14	14.88	15.62	16.36	17.12	17.85
1/2	13.60	14.44	15.30	16.16	17.00	17.85	18.70	19.55	20.40
9/16	15.30	16.26	17.22	18.18	19.14	20.08	21.02	22.00	22.95
5/8	17.00	18.06	19.13	20.19	21.25	22.32	23.38	24.44	25.50
11/16	18.70	19.86	21.04	22.21	23.38	24.54	25.70	26.88	28.05
3/4	20.40	21.68	22.96	24.23	25.50	26.78	28.05	29.33	30.60
13/16	22.10	23.48	24.86	26.24	27.62	29.00	30.40	31.76	33.15
7/8	23.80	25.30	26.78	28.26	29.75	31.24	32.72	34.21	35.70
15/16	25.50	27.10	28.69	30.28	31.88	33.48	35.06	36.66	38.25
1	27.20	28.90	30.60	32.30	34.00	35.70	37.40	39.10	40.80
1 1/16	28.90	30.70	32.52	34.32	36.12	37.92	39.74	41.54	43.35
1 1/8	30.60	32.52	34.43	36.34	38.25	40.17	42.08	44.00	45.90
1 3/16	32.30	34.32	36.34	38.36	40.38	42.40	44.42	46.44	48.45
1 1/4	34.00	36.12	38.26	40.37	42.50	44.63	46.76	48.88	51.00
1 5/16	35.70	37.93	40.16	42.40	44.64	46.86	49.08	51.32	53.55
1 3/8	37.40	39.74	42.08	44.41	46.75	49.08	51.42	53.76	56.10
1 7/16	39.10	41.54	44.00	46.44	48.88	51.32	53.76	56.21	58.65
1 1/2	40.80	43.35	45.90	48.45	51.00	53.55	56.10	58.65	61.20
1 9/16	42.50	45.16	47.82	50.48	53.14	55.78	58.42	61.10	63.75
1 5/8	44.20	46.96	49.73	52.49	55.25	58.02	60.78	63.54	66.30
1 11/16	45.90	48.76	51.64	54.51	57.38	60.24	63.10	65.98	68.85
1 3/4	47.60	50.58	53.56	56.53	59.50	62.48	65.45	68.43	71.40
1 13/16	49.30	52.38	55.46	58.54	61.62	64.70	67.80	70.86	73.95
1 7/8	51.00	54.20	57.38	60.56	63.75	66.94	70.12	73.31	76.50
1 15/16	52.70	56.00	59.29	62.58	65.88	69.18	72.46	75.76	79.05
2	54.40	57.80	61.20	64.60	68.00	71.40	74.80	78.20	81.60

Rules for Estimating the Weight of any Piece of Wrought Iron, Steel or Cast Iron

Wrought Iron.

One cubic foot of wrought iron weighs	480 lb
One square foot, one inch thick, weighs	40 lb
One square inch, one foot long, weighs	3⅓ lb

To find the weight per square foot of sheet iron, multiply the thickness in inches by 40.

To find the weight per linear foot of bars of any section, multiply the cross-sectional area in square inches by 3⅓.

Steel.

One cubic foot of steel weighs (Or just 2% more than wrought iron.)	489.6 lb
One square foot, one inch thick, weighs	40.8 lb
One square inch, one foot long, weighs	3.4 lb

To find the weight per linear foot, of bars of any section, multiply the cross-sectional area in square inches by 3.4; or, if the weight is known, the exact sectional area may be obtained by dividing by 3.4.

Cast Iron.

One cubic foot of cast iron weighs	450 lb
One square foot, one inch thick, weighs	37½ lb
One square inch, one foot long, weighs	3⅛ lb
One cubic inch weighs	0.26 lb

The weight of irregular castings must be estimated by the cubic inch.

Rules for Weights of Castings

Multiply the weight of the pattern by 18 for cast iron, 13 for brass, 19 for lead, 12.2 for tin, 11.4 for zinc; the product is the weight of the casting.

Reduction for Round Cores and Core-Prints

Rule. Multiply the square of the diameter by the length of the core in inches, and the product multiplied by 0.017 is the weight of the pine core to be deducted from the weight of the pattern.

Shrinkage in Castings

Pattern-makers' Rule

Metal	
Cast iron	⅛
Brass	3/16
Lead	⅛
Tin	1/12
Zinc	5/16

of an inch longer per linear foot

Weights of Square Cast-Iron Columns in Pounds per Linear Foot*

$a\square b$ $2a+2b$ †	Thickness of metal, inches								
	⅝ in, lb	¾ in, lb	⅞ in, lb	1 in, lb	1⅛ in, lb	1¼ in, lb	1½ in, lb	1¾ in, lb	2 in, lb
12	18.6	21.1	23.3	25.0	26.4	27.3	28.1	.....	.....
14	22.5	25.8	28.7	31.3	33.4	35.1	37.5	.....	.....
16	26.4	30.5	34.2	37.5	40.4	43.0	46.9	49.2	50.0
18	30.3	35.2	39.7	43.8	47.4	50.8	56.3	60.2	62.5
20	34.2	39.8	45.1	50.0	54.5	58.6	65.6	71.1	75.0
22	38.1	44.5	50.6	56.3	61.5	66.4	75.0	82.0	87.5
24	42.0	49.2	56.1	62.5	68.5	74.2	84.4	93.0	100.0
26	45.9	53.9	61.5	68.8	75.6	82.0	93.8	103.9	112.5
28	49.8	58.6	67.0	75.0	82.6	89.8	103.1	114.8	125.0
30	53.7	63.3	72.5	81.3	89.6	97.7	112.5	125.8	137.5
32	57.6	68.0	77.9	87.5	96.7	105.5	121.9	136.7	150.0
34	61.5	72.7	83.4	93.8	103.7	113.3	131.3	147.7	162.5
36	65.4	77.3	88.9	100.0	110.7	121.1	140.6	158.6	175.0
38	69.3	82.0	94.3	106.3	117.8	128.9	150.0	169.5	187.5
40	73.2	86.7	99.8	112.5	124.8	136.7	159.4	180.5	200.0
42	77.1	91.4	105.3	118.8	131.8	144.5	168.8	191.4	212.5
44	81.0	96.1	110.8	125.0	138.8	152.3	178.1	202.3	225.0
46	84.9	100.8	116.2	131.3	145.9	160.2	187.5	213.3	237.5
48	88.8	105.5	121.7	137.5	152.9	168.0	196.9	224.2	250.0
50	92.8	110.2	127.2	143.8	159.9	175.8	206.3	235.2	262.5
52	96.7	114.8	132.6	150.0	167.0	183.6	215.6	246.1	275.0
54	100.6	119.5	138.1	156.3	174.0	191.4	225.0	257.0	287.5
56	104.5	124.2	143.6	162.5	181.0	199.2	234.4	268.0	300.0
58	108.4	128.9	149.0	168.8	188.1	207.0	243.8	278.9	312.5
60	112.3	133.6	154.5	175.0	195.1	214.9	253.2	289.8	325.0
62	116.2	138.3	160.0	181.3	202.1	222.7	262.5	300.8	337.5
64	120.1	143.0	165.4	187.5	209.2	230.5	271.9	311.7	350.0
66	124.0	147.7	170.9	193.8	216.2	238.3	281.3	322.7	362.5
68	127.9	152.3	176.4	200.0	223.2	246.1	290.6	333.6	375.0
70	131.8	157.0	181.8	206.3	230.3	253.9	300.0	344.5	387.5
72	135.7	161.7	187.3	212.5	237.3	261.7	309.4	355.5	400.0
74	139.6	166.4	192.8	218.8	244.3	269.5	318.8	366.4	412.5
76	143.5	171.1	198.3	225.0	251.3	277.3	328.1	377.3	425.0
78	147.4	175.8	203.7	231.3	258.4	285.2	337.5	388.3	437.5
80	151.3	180.5	207.2	237.5	265.4	293.0	346.9	399.2	450.0

* Birkmire.

† a and b = either side, outside measurement. $2a+2b$ = number. Allowance has been made in this table for corners counted twice.

Example. What is the weight per linear foot of a 12 by 16 by 1 in thick column?

Solution. $2a+2b=24+32=56$. Opposite this number, under 1-in-thick metal, we find 162.5, or weight per linear foot of a column 12 by 16 by 1-in-thick.

Note. For flanges, brackets, etc., calculate the cubical contents of same and multiply by 0.26; cast iron averages 450 lb per cu ft.

Weights per Linear Foot of Circular Cast-Iron Columns * †

Outside diameter, inches	Thickness of metal, inches							
	½ in, lb	⅝ in, lb	¾ in, lb	⅞ in, lb	1 in, lb	1⅛ in, lb	1¼ in, lb	1⅜ in, lb
3	12.3	14.6	16.60	18.30	19.6	......	......	......
4	17.2	21.0	24.00	27.00	29.5	32.1	33.8	35.4
5	22.1	27.0	31.30	35.50	39.3	43.0	46.0	49.0
6	27.0	33.0	39.00	44.00	49.1	54.1	58.3	62.4
7	32.0	39.1	46.00	53.00	59.0	65.1	70.6	76.1
8	36.8	45.3	53.40	61.20	69.1	76.1	83.1	89.5
9	41.7	51.4	61.10	70.00	78.6	87.1	95.1	103.1
10	46.6	57.5	68.13	78.41	88.4	98.0	107.4	116.4
11	51.6	64.0	75.50	87.10	98.2	109.1	120.1	130.1
12	56.5	70.0	82.87	96.10	108.0	120.0	132.1	143.5
13	61.4	76.0	90.23	104.20	118.1	131.2	144.2	157.1
14	66.3	82.1	97.60	113.20	128.1	142.0	156.5	170.4
15	71.2	88.2	104.96	121.40	137.5	153.3	169.4	184.1
16	76.1	94.4	112.33	130.10	147.3	164.3	181.0	197.4
17	81.0	100.5	120.10	139.10	157.1	175.4	193.3	211.0
18	86.0	107.0	127.00	147.00	167.0	186.4	206.0	224.4
19	91.0	113.0	134.40	156.00	177.1	197.5	218.1	238.0
20	96.0	119.0	142.10	164.30	186.6	208.8	230.1	251.5
21	100.6	125.0	149.10	173.10	196.6	219.6	242.4	265.0
22	105.6	131.2	156.50	181.50	206.2	230.6	255.0	278.0
23	110.5	137.3	164.10	190.10	216.1	242.0	267.0	292.0
24	115.4	143.5	171.20	199.00	226.0	253.0	279.2	305.4

Outside diameter, inches	Thickness of metal, inches							
	1½ in, lb	1⅝ in, lb	1¾ in, lb	1⅞ in, lb	2 in, lb	2⅛ in, lb	2¼ in, lb	2⅜ in, lb
3	......	......	......	......	......	......	......	......
4	......	......	......	......	......	......	......	......
5	51.54	54.1	55.84	57.5	......	......	......	......
6	66.30	69.9	73.02	76.0	78.6	80.84	82.83	......
7	81.00	85.6	90.20	94.3	98.2	101.70	105.00	107.84
8	95.80	101.8	107.40	112.8	117.8	122.60	127.00	131.20
9	110.50	117.7	124.60	131.2	137.5	143.40	149.10	154.50
10	125.20	133.7	142.00	149.6	157.1	164.30	171.20	177.80
11	140.00	149.6	159.00	168.0	176.8	185.20	193.30	201.10
12	154.70	165.6	176.00	186.4	196.4	206.00	215.40	224.40
13	169.40	181.5	193.30	204.8	216.0	226.90	237.50	247.70
14	184.10	197.4	210.50	223.2	235.7	247.70	259.60	271.10
15	198.90	213.4	227.70	241.6	255.3	268.20	281.70	294.40
16	213.50	229.4	244.90	260.0	274.9	289.50	303.70	317.70
17	228.30	245.3	262.00	278.4	294.5	310.30	325.80	341.00
18	243.00	261.3	279.20	296.8	314.2	331.20	348.00	364.30
19	257.70	277.2	296.40	315.2	338.8	352.10	370.00	387.70
20	272.50	293.2	313.60	333.6	353.4	372.90	392.10	411.00
21	287.20	309.0	330.80	352.1	373.1	393.80	414.20	434.30
22	302.00	325.1	348.00	370.5	393.0	414.60	436.30	457.60
23	316.70	341.0	365.10	388.9	412.3	435.50	458.40	481.00
24	331.40	357.0	382.30	407.3	432.0	456.40	480.50	504.20

* Birkmire.

† The table is arranged for the weight of plain shaft. For brackets, flanges, etc., calculate the cubical contents and multiply by 0.26.

Weight of Cast-Iron Plates

Weights, in Pounds, of Cast-Iron Plates One Inch Thick

Calculated at 450 lb per cu ft

Length, inches	Width, inches									
	6 in, lb	8 in, lb	10 in, lb	12 in, lb	14 in, lb	16 in, lb	18 in, lb	20 in, lb	24 in, lb	30 in, lb
4	6.25	8.3	10.4	12.5	14.6	16.6	18.7	20.8	25	31
6	9.37	12.5	15.6	18.7	21.8	25.0	28.1	31.2	38	47
8	12.50	16.6	20.8	25.0	29.1	33.3	37.4	41.6	50	62
10	15.60	20.8	26.0	31.2	36.4	41.6	46.8	52.0	63	78
12	18.70	25.0	31.2	37.5	43.7	49.9	56.2	62.4	75	94
14	21.80	29.2	36.4	43.7	51.0	58.2	65.5	72.8	88	109
16	24.90	33.3	41.6	50.0	58.2	66.6	74.9	83.2	100	125
18	28.10	37.5	46.8	56.2	65.5	74.9	84.2	93.6	113	140
20	31.20	41.6	52.0	62.3	72.8	83.2	93.6	104.0	125	156
22	34.30	45.8	57.2	68.6	80.1	91.5	103.0	114.4	138	172
24	37.50	50.0	62.4	75.0	87.4	99.8	112.3	124.8	150	187
26	40.60	54.0	67.6	81.2	94.6	108.2	121.7	135.2	163	203
28	43.60	58.2	72.8	87.5	101.9	116.5	131.0	145.6	175	218
30	46.80	62.4	78.0	93.7	109.2	124.8	140.4	156.0	188	234
32	49.80	66.6	83.2	100.0	116.5	133.1	150.3	166.4	200	250
36	56.10	75.0	93.6	112.5	131.0	150.0	168.4	187.2	225	281

For larger plates take size of plate ONE-HALF smaller and multiply by 2. Thus a plate 28 by 32 in will weigh twice as much as one 14 by 32 in. For plates more or less than one inch in thickness multiply weight of plate by thickness in inches.

Approximate Weights of Square-Ribbed Cast-Iron Column-Bases

The following table, giving the weight of cast-iron column-bases, will be useful when estimating the steel and iron in tall buildings.*

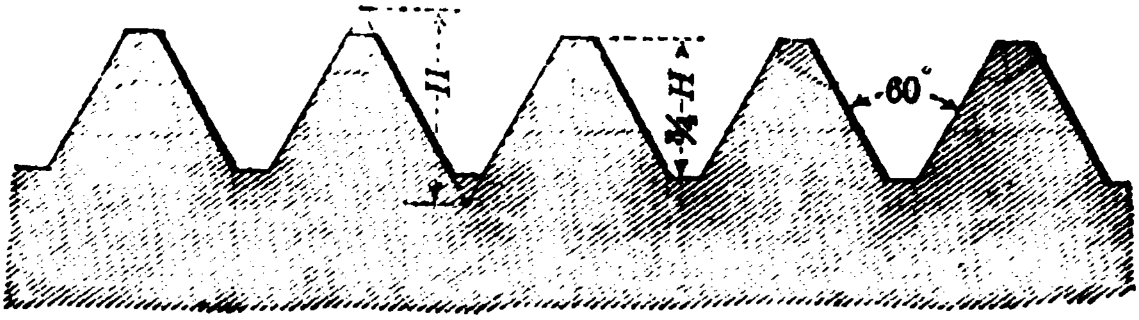
Size of square base, in	Weight, lb	Size of square base, in	Weight, lb
22×22	600	32×32	1 340
24×24	750	34×34	1 450
26×26	880	36×36	1 600
28×28	1 020	38×38	1 720
30×30	1 180	40×40	1 850

* H. G. Tyrrell, in Architects and Builders Magazine, January, 1903.

Screw-Threads, Nuts, and Bolt-Heads

Standard Screw-Threads

Recommended by Franklin Institute, December 15, 1864, and adopted by Navy Department of the United States; by the R. R. Master Mechanics' and Master Car-Builders' Associations; by Jones & Laughlin Steel Company; and by many other of the prominent engineering and mechanical establishments of the country.



Angle of thread 60°. Flat at top and bottom ⅛ of pitch.

Diam of screw, in	Threads per inch	Diam at root of thread, in	Area at root of thread, sq in	Diam of screw, in	Threads per inch	Diam at root of thread, in	Area at root of thread, sq in
¼	20	0.185	0.027	2	4½	1.712	2.302
5/16	18	0.240	0.045	2¼	4½	1.962	3.023
⅜	16	0.294	0.068	2½	4	2.176	3.719
7/16	14	0.344	0.093	2¾	4	2.426	4.620
½	13	0.400	0.126	3	3½	2 629	5.428
9/16	12	0.454	0.162	3¼	3½	2.879	6.510
⅝	11	0.507	0.202	3½	3¼	3.100	7.548
¾	10	0.620	0.302	3¾	3	3.317	8.641
⅞	9	0.731	0.420	4	3	3.567	9.963
1	8	0.837	0.550	4¼	2⅞	3.798	11.329
1⅛	7	0.940	0.694	4½	2¾	4.028	12.753
1¼	7	1.065	0.893	4¾	2⅝	4.256	14.226
1⅜	6	1.160	1.057	5	2½	4.480	15.763
1½	6	1.284	1.295	5¼	2½	4.730	17.572
1⅝	5½	1.389	1.515	5½	2⅜	4.953	19.267
1¾	5	1.491	1.746	5¾	2⅜	5.203	21.262
1⅞	5	1 616	2.051	6	2¼	5.423	23.098

Nuts and Bolt-Heads are determined by the following rules, which apply to both square and hexagon nuts:

Short diameter of rough nut = 1½ × diam of bolt + ⅛ in.
Short diameter of finished nut = 1½ × diam of bolt + 1/16 in.
Thickness of rough nut = diam of bolt.
Thickness of finished nut = diam of bolt − 1/16 in.
Short diameter of rough head = 1½ × diam of bolt + ⅛ in.
Short diameter of finished head = 1½ × diam of bolt + 1/16 in.
Thickness of rough head = ½ short diam of head.
Thickness of finished head = diam of bolt − 1/16 in.

The long diameter of a hexagon nut may be determined by multiplying the short diameter by 1.155, and the long diameter of a square nut by multiplying the short diameter by 1.414.

Standard Dimensions of Nuts and Bolt-Heads

Diam of bolt	Short diam, rough	Short diam, finished	Long diam, rough	Long diam, rough	Thickness, rough. Nut	Thickness, finished. Both	Thickness, rough. Head
1/4	1/2	7/16	37/64	7/10	1/4	3/16	1/4
5/16	19/32	17/32	11/16	19/12	5/16	1/4	19/64
3/8	11/16	5/8	51/64	63/64	3/8	5/16	11/32
7/16	25/32	23/32	9/10	1 7/64	7/16	3/8	25/64
1/2	7/8	13/16	1	1 15/64	1/2	7/16	7/16
9/16	31/32	29/32	1 1/8	1 23/64	9/16	1/2	31/64
5/8	1 1/16	1	1 7/32	1 1/2	5/8	9/16	17/32
3/4	1 1/4	1 3/16	1 7/16	1 49/64	3/4	11/16	5/8
7/8	1 7/16	1 3/8	1 21/32	2 1/32	7/8	13/16	23/32
1	1 5/8	1 9/16	1 7/8	2 19/64	1	15/16	13/16
1 1/8	1 13/16	1 3/4	2 3/32	2 9/16	1 1/8	1 1/16	29/32
1 1/4	2	1 15/16	2 5/16	2 53/64	1 1/4	1 3/16	1
1 3/8	2 3/16	2 1/8	2 17/32	3 3/32	1 3/8	1 5/16	1 3/32
1 1/2	2 3/8	2 5/16	2 3/4	3 23/64	1 1/2	1 7/16	1 3/16
1 5/8	2 9/16	2 1/2	2 31/32	3 5/8	1 5/8	1 9/16	1 9/32
1 3/4	2 3/4	2 11/16	3 3/16	3 57/64	1 3/4	1 11/16	1 3/8
1 7/8	2 15/16	2 7/8	3 13/32	4 5/32	1 7/8	1 13/16	1 15/32
2	3 1/8	3 1/16	3 5/8	4 27/64	2	1 15/16	1 9/16
2 1/4	3 1/2	3 7/16	4 1/16	4 61/64	2 1/4	2 3/16	1 3/4
2 1/2	3 7/8	3 13/16	4 1/2	5 31/64	2 1/2	2 7/16	1 15/16
2 3/4	4 1/4	4 3/16	4 29/32	6	2 3/4	2 11/16	2 1/8
3	4 5/8	4 9/16	5 3/8	6 17/32	3	2 15/16	2 5/16
3 1/4	5	4 15/16	5 13/16	7 1/16	3 1/4	3 3/16	2 1/2
3 1/2	5 3/8	5 5/16	6 7/64	7 39/64	3 1/2	3 7/16	2 11/16
3 3/4	5 3/4	5 11/16	6 21/32	8 1/8	3 3/4	3 11/16	2 7/8
4	6 1/8	6 1/16	7 3/32	8 41/64	4	3 15/16	3 1/16
4 1/4	6 1/2	7 7/16	7 9/16	9 3/16	4 1/4	4 3/16	3 1/4
4 1/2	6 7/8	6 13/16	7 31/32	9 3/4	4 1/2	4 7/16	3 7/16
4 3/4	7 1/4	7 3/16	8 13/32	10 1/4	4 3/4	4 11/16	3 5/8
5	7 5/8	7 9/16	8 27/32	10 49/64	5	4 15/16	3 13/16
5 1/4	8	7 15/16	9 9/32	11 23/64	5 1/4	5 3/16	4
5 1/2	8 3/8	8 5/16	9 23/32	11 7/8	5 1/2	5 7/16	4 3/16
5 3/4	8 3/4	8 11/16	10 3/32	12 3/8	5 3/4	5 11/16	4 3/8
6	9 1/8	9 1/16	10 19/32	12 15/16	6	5 15/16	4 9/16

Weights of One Hundred Bolts With Square Heads and Nuts

INCLUDES WEIGHT OF NUT

Hoopes & Townsend's List

Length under head to point in	Diameter of bolts								
	¼ in,	5/16 in,	⅜ in,	7/16 in,	½ in, lb	⅝ in, lb	¾ in, lb	⅞ in, lb	1 in, lb
	lb	lb	lb	lb					
1½	4.00	7.00	10.50	15.20	22.50	39.50	63.00	.....	.....
1¾	4.40	7.50	11.25	16.30	23.82	41.62	66.00	.....	.....
2	4.75	8.00	12.00	17.40	25.15	43.75	69.00	109.00	163
2¼	5.15	8.50	12.75	18.50	26.47	45.88	72.00	113.25	169
2½	5.50	9.00	13.50	19.60	27.80	48.00	75.00	117.50	174
2¾	5.75	9.50	14.25	20.70	29.12	50.12	78.00	121.75	180
3	6.25	10.00	15.00	21.80	30.45	52.25	81.00	126.00	185
3½	7.00	11.00	16.50	24.00	33.10	56.50	87.00	134.25	196
4	7.75	12.00	18.00	26.20	35.75	60.75	93.10	142.50	207
4½	8.50	13.00	19.50	28.40	38.40	65.00	99.05	151.00	218
5	9.25	14.00	21.00	30.60	41.05	69.25	105.20	159.55	229
5½	10.00	15.00	22.50	32.80	43.70	73.50	111.25	168.00	240
6	10.75	16.00	24.00	35.00	46.35	77.75	117.30	176.60	251
6½	.....	.....	25.50	37.20	49.00	82.00	123.35	185.00	262
7	.....	.....	27.00	39.40	51.65	86.25	129.40	193.65	273
7½	.....	.....	28.50	41.60	54.30	90.50	135.00	202.00	284
8	.....	.....	30.00	43.80	59.60	94.75	141.50	210.70	295
9	.....	.....	.....	46.00	64.90	103.25	153.60	227.75	317
10	.....	.....	.....	48.20	70.20	111.75	165.70	224.80	339
11	.....	.....	.....	50.40	75.50	120.25	177.80	261.85	360
12	.....	.....	.....	52.60	80.80	128.75	189.90	278.90	382
13	.....	.....	.....	.....	86.10	137.25	202.00	295.95	404
14	.....	.....	.....	.....	91.40	145.75	214.10	313.00	426
15	.....	.....	.....	.....	96.70	154.25	226.20	330.05	448
16	.....	.....	.....	.....	102.00	162.75	238.30	347.10	470
17	.....	.....	.....	.....	107.30	171.00	250.40	364.15	492
18	.....	.....	.....	.....	112.60	179.50	262.60	381.20	514
19	.....	.....	.....	.....	117.90	188.00	274.70	398.25	536
20	.....	.....	.....	.....	123.20	206.50	286.80	415.30	558
Per inch additional	1.37	2.13	3.07	4.18	5.45	8.52	12.27	16.70	21.82

Weights of Nuts and Bolt-Heads, in Pounds

For calculating the weight of longer bolts

Diameter of bolt, in inches		¼	⅜	½	⅝	¾	⅞
Weight of hexagon nut and head...	....	0.017	0.057	0.128	0.267	0.43	0.73
Weight of square nut and head....	....	0.021	0.069	0.164	0.320	0.55	0.88
Diameter of bolt, in inches	**1**	**1¼**	**1½**	**1¾**	**2**	**2½**	**3**
Weight of hexagon nut and head...	1.10	2.14	3.78	5.6	8.75	17	28.8
Weight of square nut and head....	1.31	2.56	4.42	7.0	10.50	21	36.4

Weights of Rivets and Round-Headed Bolts Without Nuts. Steel

POUNDS PER HUNDRED

Length, in	⅜ in diam	½ in diam	⅝ in diam	¾ in diam	⅞ in diam	1 in diam	1⅛ in diam	1¼ in diam
1¼	5.5	12.8	22.0	29.3	43.9	66.6	93.3	127
1½	6.3	14.2	24.1	32.4	48.2	72.1	100	136
1¾	7.0	15.5	26.3	35.5	52.5	77.7	107	145
2	7.9	16.9	28.5	38.7	56.7	83.3	114	153
2¼	8.7	18.3	30.7	41.8	61.0	88.8	121	162
2½	9.4	19.7	32.8	44.9	65.2	94.4	128	171
2¾	10.2	21.1	35.0	48.0	69.5	100.	136	179
3	11.0	22.5	37.2	51.1	73.7	105.	143	188
3¼	11.7	23.9	39.3	54.3	78.0	111	150	197
3½	12.6	25.3	41.5	57.4	82.3	116	157	205
3¾	13.4	26.7	43.7	60.5	86.5	122	164	214
4	14.1	28.1	45.9	63.6	90.8	128	170	223
4¼	14.9	29.4	48.0	66.7	95.0	134	177	231
4½	15.7	30.8	50.2	69.9	99.3	139	185	240
4¾	16.5	32.2	52.4	73.0	104	145	192	249
5	17.2	33.6	54.5	76.1	108	150	199	258
5¼	18.1	35.0	56.7	79.2	112	156	206	266
5½	18.8	36.4	58.9	82.3	116	161	213	275
5¾	19.6	37.8	61.1	85.5	120	166	220	284
6	20.4	39.2	63.2	88.6	124	172	227	292
6½	21.9	42.0	67.6	95.1	133	184	241	310
7	23.5	44.7	71.9	101	142	195	255	327
7½	25.1	47.5	76.1	108	150	206	269	345
8	26.6	50.3	80.6	114	159	217	284	362
8½	28.2	53.1	85.0	120	167	227	298	379
9	29.8	55.9	89.3	126	176	239	312	397
9½	31.3	58.7	93.7	133	185	250	325	414
10	32.8	61.4	98.0	139	193	261	340	431
10½	34.5	64.2	103	145	202	272	354	449
11	36.0	67.0	107	151	210	284	368	466
11½	37.6	69.8	111	158	218	295	382	484
12	39.2	72.5	115	164	227	306	396	501
Heads......	1.8	5.8	11.1	13.6	22.6	39.0	58.0	83.5

For length of shaft required to form rivet-head, see Table IV, page 420.

NAILS AND SCREWS*

Nails. Based upon the process of manufacture there are three kinds of nails in common use, namely, plate or cut nails, wire nails, and clinch-nails. These are briefly described in the following subdivisions of this article and other data bearing on the subject is included.

(1) **Cut Nails.** Cut nails are made from a strip of rolled iron or steel of the same thickness as the finished nail and a little wider than its length, the fiber of the iron being parallel with the length of the nail. Special machinery cuts the nails out in alternate wedge-shaped slices, the heads are then stamped on them and the finished nails dropped into the casks. Cut nails made from iron are generally preferred for use in exposed positions. Cut nails are made in a variety of shapes to suit special uses. For ordinary use in building, nails of three different shapes are made, and the nails are called COMMON NAILS, FINISH-NAILS and CASING-NAILS. The common nails are used for rough work, finish-nails for finished work, and casing-nails for flooring, matched ceiling and sometimes for pine casings, although the heads are rather too large for finish-work. Cut nails are beginning to return to favor as they have holding power and lasting qualities superior to wire nails.

(2) **Brads.** Brads are thin nails with a small head, used for small finish, panel-moldings, etc. They vary from ¼ to 2 in in length.

(3) **Clout-Nails.** Clout-nails are made with broad, flat heads, and are sold in sizes varying from ⅜ to 2½ in in length. They are used chiefly for fastening gutters and metal-work. Special nails are also made for lathing, slating, shingling, etc.

(4) **Wire Nails.** These have of late years become as common as the cut nails, and are sold at about the same price. They are said to be stronger for driving than the cut nails, not so liable to bend or break, especially when driven into hard woods, and less liable to split the wood; for these reasons they are generally preferred by carpenters. Wire nails are made from wire, of the same section-diameter as the shank of the nail, by a machine which cuts the wire in even lengths, heads and points them, and, when desired, also barbs them. In general the same classification is used for cut nails. It should be noticed that the gauge of the wire and the shape of the head vary in the different kinds, and that some are barbed, others plain. The various types of wire nails are drawn ROUND, SMOOTH or BARBED, for the domestic trade; for export they are drawn OVAL, SQUARE, or DIAMOND-SHAPED, according to the country to which they are to be shipped and its requirements. It is customary to charge 15 cents more per 100 lb for standard nails, BARBED, than for the same nails, SMOOTH.

(5) **Clinch-Nails.** These are made from open-hearth or Bessemer-steel wire. Any ordinary wire nail will clinch, especially when made with DUCK-BILL or flattened points for clinching purposes, or even otherwise, if annealed. These nails are used only in places where it is desired to turn over the ends of the nails to form a clinch, as in the case of battens or cleats.

(6) **Length and Weight of Nails.** The length of nails is designated by PENNIES (*d's*). Two explanations are given for the origin of this classification; for example, that tenpenny nails originally sold for tenpence a hundred, or that 1000 tenpenny nails originally weighed 10 lb. The designation is retained by manufacturers, both for cut and wire nails. The weights expressed in pennies run from two pennies to sixty pennies, the larger sizes being designated by

* Condensed from article by Thomas Nolan in chapter on Builders' Hardware in revised edition of Building Construction and Superintendence, Part II, Carpenters' Work, by F. E. Kidder.

fractions of an inch. The sizes and lengths of various kinds of nails and tacks are given in tables on pages 1531 to 1534.

(7) **Sizes of Nails for Different Classes of Work.** It is imperative for first-class work that nails of proper size should be used and to insure the best results it is well in certain classes of work to specify the sizes which are to be used. For framing, twentypenny, fortypenny and sixtypenny nails, or spikes, are used, according to the size of the timber. For sheathing and roof-boarding, under-floors and cross-bridging, tenpenny common nails should be used. For over-floors tenpenny floor-nails or casing-nails should be used for jointed boards, and ninepenny or tenpenny for matched flooring, although eightpenny nails are sometimes used. Ceiling when ¾ in thick is generally put up with eightpenny casing-nails, and when thinner stuff is used, with sixpenny nails. For inside finish any size of finish-nails or brads from eightpenny down to twopenny is used, according to the thickness and size of the moldings. For pieces exceeding 1 in in thickness, tenpenny nails should be used. Clapboarding is generally put on with sixpenny finish-nails or casing-nails. Threepenny to fourpenny are used for shingling and slating, and threepenny for lathing. For slating, galvanized nails should be used, and they are also better for shingling. Whether wire or cut nails should be used may generally be left to the builder; but in places where there is any danger of the nails being drawn out either by the warping of the boards or from the pull of the nail, cut nails should be used, as they have greater holding power than the wire nails under certain conditions. In regard to the comparative holding power of cut nails and wire nails, and barbed nails and smooth nails, and tests made to determine this property, see page 1531.

(8) **Copper and Brass Nails.** Nails are also made of copper and cast brass, and these are sometimes used in connection with boat-building, refrigerator-work, etc. One wing of the Physical Laboratory Building of Harvard College is put together entirely with brass and copper. As the rooms were intended for use in delicate electrical work, no iron was used in their construction.

(9) **Cement-coated Wire Nails.** The coating consists of various resinous gums mixed by a secret formula, and put on the nails by a baking-process which involves the use of quite complicated machinery. Although the chief market for coated nails is among the users of packages to be shipped, there is a limited market for them among builders, for construction-purposes. The chief merit of the coating is that it gives the nail an adhesive resistance approximately twice that of ordinary wire nails. This quality appeals especially to the manufacturers and users of packages to be shipped, for which strength is particularly wanted. It is desirable for construction-purposes also, but the lack of holding power in plain wire nails is not so apparent in building. About 90% of the output goes to box-factories and large shippers. Cement-coated nails are quite widely used, also, in laying both ordinary and parquetry-flooring. The use of these nails, with a special head which leaves a small hole, gives a firm floor and prevents springing. Though the makers do not claim that the nails are absolutely rust-proof, they do claim that nails thus treated will resist the effects of moisture from 20 to 50% better than the uncoated wire nails. But it is when in use that the non-rusting quality is most evident. There is more coating on the nails than is actually necessary for holding power. The heat caused by the friction of driving the nail softens the coating and the surplus is forced toward the head, completely closing the opening; this prevents the admission of moisture between the wood and the nail. Under similar conditions, the life of a cement-coated nail will be about twice as long as that of an uncoated one. Less force is needed to drive a coated nail as the softened coating forms a lubricant. These nails are made in

wo types, differing only in the heads, and are either COOLERS or SINKERS. he former have large flat heads; the latter, heads slightly reinforced by counter-nking. They are made to replace common nails, in sizes from ¼ in to 1 in, d are used for framing, boarding, shingling and staging, and for boxes and ates. Results of tests made with cement-coated nails to determine their lhesive resistance in comparison with the common smooth-wire nails are given low.

The following table shows the result of tests made at the United States Arsenal, atertown, Mass., in 1902, the wood being pine:

Comparative Adhesive Resistance of Common Smooth-Wire Nails and Cement-Coated Nails

All nails were driven into the same piece and were perpendicular to the grain

Size and name	Diameter, in	Length driven,* in	Adhesive resistance,† lb
enpenny, common, smooth	0.145	2½	167
enpenny, coated	0.117	2½	418
inepenny, common, smooth	0.132	2¼	182
inepenny, coated	0.114	2¼	327
ightpenny, common, smooth	0.132	2	189
ightpenny, coated	0.112	2	316
ixpenny, common, smooth	0.097	1⅝	106
ixpenny, coated	0.092	1⅝	226

* All of the nails were left with their heads projecting from ¼ to ½ in.
† Average of three trials.

Holding Power of Nails. A committee appointed by the Wheeling nail-nufacturers, a number of years ago, to test the comparative holding power of and wire nails, published the following data. The kind of wood is not named. e effect of barbs is slight, and definite conclusions await complete tests.

Pounds Required to Pull Nails Out

	Cut	Wire		Cut	Wire
ventypenny	1 593	703	Sixpenny	383	200
npenny	908	315	Fourpenny	286	123
ghtpenny	597	227			

he holding power of nails varies with the kind of wood into which they are en. Austin T. Byrne gives the relative holding power of woods as ABOUT as ws: White pine, 1; yellow pine, 1.5; white oak, 3; chestnut, 1.6; beech, 3.2; more, 2; elm, 2; basswood, 1.2.

Comparative Holding Power of Cut and Wire Nails

ry thorough tests of the comparative holding power of wire nails and cut OF EQUAL LENGTHS AND WEIGHTS were made at the U. S. Arsenal in 1892 and . From forty series, comprising forty sizes of nails driven in spruce wood, it found that the cut nails showed an average superiority of 60.50%, the com-nails showing an average superiority of 47.51% and the finishing-nails an ge of 72.22%. In eighteen series, comprising six sizes of BOX-NAILS driven pine wood, in three ways the cut nails showed an average superiority of %. In no series of tests did the wire nails hold as much as the cut nails.

Quantity of Nails Required for Different Kinds of Work

For 1 000 shingles* allow 3½ to 6½ lb fourpenny nails or 3½ to 4½ lb threepen
1 000 laths, 7 lb threepenny fine, or for 100 sq yd of lathing, 10 lb threepenny fi
1 000 sq ft of beveled siding, 18 lb sixpenny
1 000 sq ft of sheathing, 20 lb eightpenny or 25 lb tenpenny
1 000 sq ft of flooring, 30 lb eightpenny or 40 lb tenpenny
1 000 sq ft of studding, 15 lb tenpenny and 5 lb twentypenny
1 000 sq ft of 1 by 2½-in furring, 12-in centers, 9 lb eightpenny or 14 lb tenpen
1 000 sq ft of 1 by 2½-in furring, 16-in centers, 7 lb eightpenny or 10 lb tenpen

* Depends upon width and length of shingles and kind of nails.

Cut Steel Nails and Spikes

Sizes, lengths, and approximate number per pound
Taken from the Handbook of the Cambria Steel Company

Sizes	Length, inches	Common	Clinch	Finishing	Casing and box	Fencing	Spikes
2d	1	740	400	1 100	....	....	....
3d	1¼	460	260	880	....	....	....
4d	1½	280	180	530	420	....	....
5d	1¾	210	125	350	300	100	....
6d	2	160	100	300	210	80	....
7d	2¼	120	80	210	180	60	....
8d	2½	88	68	168	130	52	....
9d	2¾	73	52	130	107	38	....
10d	3	60	48	104	88	26	....
12d	3¼	46	40	96	70	20	....
16d	3½	33	34	86	52	18	17
20d	4	23	24	76	38	16	14
25d	4¼	20	....	....	....	....	....
30d	4½	16½	....	....	30	....	11
40d	5	12	....	....	26	....	9
50d	5½	10	....	....	20	....	7½
60d	6	8	....	....	16	....	6
......	6½	....	....	....	....	....	5½
......	7	....	....	....	....	....	5

Sizes	Length, inches	Barrel	Light barrel	Slating	Tobacco	Brads	Shingle
....	⅝	750	....	....			
....	¾	600	....	....			
....	⅞	500	....	....			
2d	1	450	....	340			
....	1⅛	310	400	....			
3d	1¼	280	304	280			
....	1⅜	210	....	....			
4d	1½	190	224	220			
5d	1¾	....	....	180	130	......	......
6d	2	....	....	....	97	120	......
7d	2¼	....	....	....	85	94	......
8d	2½	....	....	....	68	74	90
9d	2¾	....	....	....	58	62	72
10d	3	....	....	....	48	50	60
12d	3¼	....	....	....	....	40	......
16d	3½	....	....	....	....	27	......

Sizes	Length, inches	Flat grip, fine	Edge grip, fine
......	¾	1 462	......
......	⅞	1 300	......
2d	1	1 100	960
3d	1⅛	800	750
4d	1⅜	650	600

Steel-Wire Nails, Spikes, and Tacks

SIZE, LENGTH, GAUGE AND APPROXIMATE NUMBER TO THE POUND

Compiled from Catalogue of American Steel and Wire Company, 1910
American Steel and Wire Company's gauge. (See page 1512.)

Common nails and brads *				Casing-nails †		Finishing-nails†	
Size	Length, in	Gauge	Number to pound	Gauge	Number to pound	Gauge	Number to pound
2d	1	15	876	15½	1 010	16½	1 351
3d	1¼	14	568	14½	635	15½	807
4d	1½	12½	316	14	473	15	584
5d	1¾	12½	271	14	406	15	500
6d	2	11½	181	12½	236	13	309
7d	2¼	11½	161	12½	210	13	238
8d	2½	10¼	106	11½	145	12½	189
9d	2¾	10¼	96	11½	132	12½	172
10d	3	9	69	10½	94	11½	121
12d	3¼	9	63	10½	87	11½	113
16d	3½	8	49	10	71	11	90
20d	4	6	31	9	52	10	62
30d	4½	5	24	9	46	....	....
40d	5	4	18	8	35	....	....
50d	5½	3	14				
60d	6	2	11				

Shingle-nails			
Size	Length, in	Gauge	Number to pound
3d	1¼	13	429
3½d	1⅜	12½	345
4d	1½	12	274
5d	1¾	12	235
6d	2	12	204
7d	2¼	11	139
8d	2½	11	125
9d	2¾	11	114
10d	3	10	83
Fine nails			
2d	1	16½	1 351
3d	1⅛	15	778
4d	1½	14	473
2d extra fine	1	17	1 560
3d extra fine	1⅛	16	1 015

Spikes ‡			
Size	Length, in	Gauge	Number to pound
10d	3	6	41
12d	3¼	6	38
16d	3½	5	30
20d	4	4	23
30d	4½	3	17
40d	5	2	13
50d	5½	1	10
60d	6	1	9
7″	7	5/16″	7
8″	8	3/8″	4
9″	9	3/8″	3½
10″	10	3/8″	3
12″	12	3/8″	2½

* Common brads differ from common nails only in the head and point.
† Lengths are the same as common nails for corresponding size.
‡ Spikes are made with chisel-points and diamond points; also with convex heads and
heads.

Steel-Wire Nails (Continued)

Clinch-nails				Fence-nails *		Slating-nails *	
Size	Length, in	Gauge	Number to pound	Gauge	Number to pound	Gauge	Numbe[r] to pound
2d	1	14	710			12	411
3d	1¼	13	429	No 5 smallest size		10½	225
4d	1½	12	274			10½	187
5d	1¾	12	235	10	142	10	142
6d	2	11	157	10	124	9	103
7d	2¼	11	139	9	92	Barbed roofing-nails †	
8d	2½	10	99	9	82		
9d	2¾	10	90	8	62	¾"×No 13	714
10d	3	9	69	7	50	⅞"×No 12	469
12d	3¼	9	62	6	40	1"×No 12	411
16d	3½	8	49	5	30	1⅛"×No 12	365
20d	4	7	37	4	23	1¼"×No 11	251

* Length same as clinch-nails of corresponding size.

† Roofing-nails are designated by the length, not by PENNY. These nails are made i[n] lengths up to 2 in.

Wire Tacks

Title, ounce	Length, in	Number per pound	Title, ounce	Length, in	Number per pound	Title, ounce	Length, in	Numbe[r] per pound
1	⅛	16 000	4	7/16	4 000	14	13/16	1 143
1½	3/16	10 666	6	9/16	2 666	16	⅞	1 000
2	¼	8 000	8	⅝	2 000	18	15/16	[illegible]
2½	5/16	6 400	10	11/16	1 600	20	1	800
3	⅜	5 333	12	¾	1 333	22	1 1/16	727
......	......	....	......	......	....	24	1⅛	666

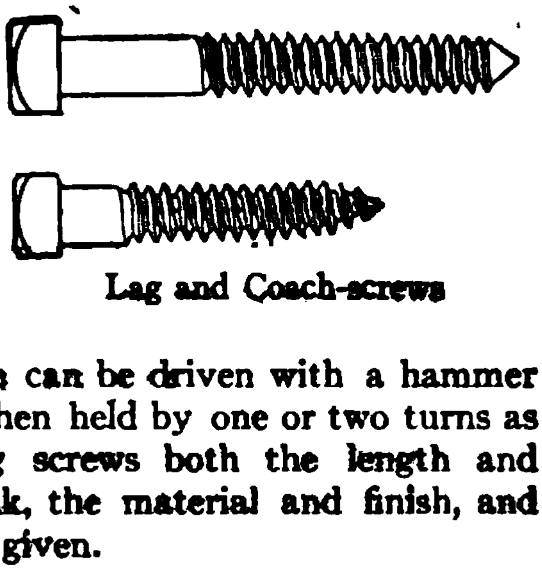
Wire carpet-tacks are made polished, blued, tinned, or coppered; there are also upho[l]sterers' and bill-posters' or railroad tacks.

Expansion-bolt

Expansion-Bolts. These are commonly used for boltin[g] wood or iron to masonry that is already built. A hole i[s] drilled in the masonry of such size that the expansion-nut wi[ll] fit closely, and when the bolt is screwed up the nut expand[s] and binds firmly in the masonry. The illustration shows th[e] Evans expansion-bolt, which is also furnished with screw-hea[d] bolts. There are other forms of expansion-bolts on the marke[t]. From experiments on expansion-bolts it was found that th[e] holding capacity was 264 lb per sq in when embedded in 1: Portland-cement mortar, 843 per sq in when embedded i[n] sulphur and 485 lb per sq in when embedded in lead. Fo[r] average working unit-stresses it is safe to use about one-fifth o[f] the values given. When the work is exposed to rain or moisture sulphur shoul[d]

ot be used as the acid which results will rust the metal and will also tend to lisintegrate the masonwork at the point of entrance of the bolt.

Screws. The substitution of screws for nails in building operations is a narked feature of modern work. Trimming hardware of all descriptions is put n with screws, and a great deal of panel-work, inside finish, etc., is put together vith them. Stop-beads, the casings of plumbing-fixtures, etc., should be fastened with screws, as well as all kinds of store and office-fixtures, and cabinet-work in general, except where the joints are glued. Screws are also largely used n making furniture. They present a neater appearance than nails, have greater olding power and are less apt to injure the material if it should be removed and eplaced. By making holes for the screws with a bit, all danger of splitting the nish is averted. The ordinary type of screw has a gimlet-point by which it can e turned into the wood without the aid of a bit. The heads are made in various rms to suit different uses. Screws are made ordinarily of steel, but sometimes f brass and bronze. The latter sort are used for screwing in place finished hardare of the same material, and have heads finished to correspond with the trimings. Steel screws, also, are finished with blue, bronze, lacquered, galvanized, r tinned surface, to match the cheaper class of trimmings. The galvanized nish is used in building operations at the seashore. Screws with blue surface, alled BLUED SCREWS, are generally used with japanned hardware and for stopads, and wherever a cheap round-headed screw is desired. Silver, nickel, and old-plated screws are also manufactured for use in connection with similar ardware. Steel screws for wood are made in twenty different lengths, varying om ¼ to 6 in, and each length of screw has from six to eighteen varieties in ickness, there being in all thirty-one different gauges; so that altogether there are the market about two hundred and fifty fferent sizes of ordinary screws used for oodwork. The most common shapes are e ordinary flat head, round head and oval ad. The oval-head screw is tapered for untersinking but is slightly rounded on top. tent diamond-point steel screws are made oecially for driving with a hammer. These can be driven with a hammer eir entire length into any hard wood, and then held by one or two turns as curely as the ordinary screw. In ordering screws both the length and mber of the gauge or diameter of the shank, the material and finish, and e use to which they are to be put, should be given.



Lag and Coach-screws

Screws for Metal have the same diameter throughout and the threads are V-aped.

Sizes of Screws. The sizes of screws are given in length in inches and the mber of the gauge, the gauge denoting the diameter. Thus, a 1-in No. 12 screw t in long and 0.2158 in in diameter. The gauge-numbers range from 0 to 30 d the lengths from ¼ to 6 in. The lengths vary by eighths of an inch up to n, by quarters of an inch up to 3 in and by halves of an inch up to 5 in. Screws m ⅝ to 4½ in long are made in about sixteen different gauge-numbers. Table II, page 402, gives the diameter to four places in decimals of an inch of the erican screw-gauge. It should be noticed that, unlike the ordinary wireges, the 0 of the screw-gauge indicates the diameter of the smallest screw ile the diameter of the screw increases with the number of the gauge.

Lag-Screws and Coach-Screws are large, heavy screws used where great ength is required, as in heavy framing, and for fixing ironwork to timber.

Lag-screws with conical point are made with diameters of 5/16, 3/8, 7/16, 1/2, 9/16, 5/8, 3/4, and 1 in, and in lengths from 1 1/2 to 12 in; coach-screws in diameters from 1/4 to 3/4 in and in lengths from 1 1/2 to 12 in. For putting in lag-screws a hole should be bored which has a diameter a little greater than the unthreaded shank of the screw and it should be bored to a depth corresponding to the length of the unthreaded shank. A second hole should then be bored at the bottom of the first hole of a diameter somewhat less than that of the threaded shank and to a depth of about half its length.

Holding Power of Lag-Screws

Tests made by A. J. Cox, University of Iowa, 1891, quoted by Kent, page 324

Kind of wood	Size of screw, in	Size of hole bored, in	Length in wood, in	Maximum resistance, lb	Number of tests
Seasoned white oak	5/8	1/2	4 1/2	8 037	3
Seasoned white oak	9/16	7/16	3	6 480	1
Seasoned white oak	1/2	3/8	4 1/2	8 780	2
Yellow-pine stick	5/8	1/2	4	3 800	2
White cedar, unseasoned	5/8	1/2	4	3 405	2

Hoopes & Townsend give the force required to draw screws out of yellow pine as follows:

Screw	1/2 in	5/8 in	3/4 in	7/8 in	1 in
Wood, depth	3 1/2 in	4 in	4 in	5 in	6 in
Force, pounds	4 960	6 000	7 685	11 500	12 620

Wooden-screws are sold by the gross, lag-screws and coach-screws by the pound.

DATA ON EXCAVATING *

Excavating is almost invariably measured by the cubic yard of 27 cu ft. For measuring excavations of irregular depth see page 65. For computing the contents of wells and cesspools, the circular area in square feet may be obtained from the table on page 51, and this circular area multiplied by the depth in feet will give the contents in cubic feet. The cost of excavating and removing earth is ordinarily made up of the following items:

(1) Loosening the earth for the shovelers;
(2) Loading by shovels into carts or barrows;
(3) Hauling or wheeling it away, including emptying and returning;
(4) Spreading it out on the dump;

For every large job, such as railroad-work, it is also necessary to make an allowance for keeping the hauling-road in repair, for sharpening and repair of tools, and for carts, harness, superintendence and water-carriers. Where the dirt excavated can be spread over the ground immediately surrounding the excavation the loosened dirt may be removed by scrapers without shoveling.

Data for Estimating Cost of Loosening Earth. Two men with a plough and team of horses will loosen from 20 to 30 cu yd of strong, heavy soil per hour or

* All prices given are pre-war prices and are retained for purposes of comparison of relative values.

om 40 to 60 cu yd of ordinary loam. One man with a pick will loosen 1⅛ yd er hour of stiff clay or cemented gravel, 4 yd of common loam, or 6 yd of light and.

The average quantity of LOOSENED EARTH that a man can shovel into a cart er hour is:

Loam or sand	2.0 cu yd
Clay and heavy soils	1.7 cu yd
Rock	1.0 cu yd

Average earth when loosened swells to from 1⅛ to 1¼ times its original bulk n place.

The capacity of vehicles used for moving excavated materials is about as ollows:

Wheelbarrows	3 to 4 cu ft
One-horse dump-carts	18 to 22 cu ft
Two-horse dump-wagons	27 to 45 cu ft *
Drag-scrapers	3 to 7 cu ft
Wheel-scrapers	10 to 17 cu ft
Dump-cars on rails	27 to 80 cu ft

The Economical Length of Haul with drag-scrapers is about 150 ft; with heeled scrapers, 500 ft; with wheelbarrows, 250 ft; with one-horse dump-carts, 0 ft.† The average speed of horses is given as about 200 ft per minute. Much valuable data for estimating ‡ the cost of excavating may be found in e Civil Engineer's Handbooks.

Weight of Earth, Sand and Gravel. For general calculations the following erage values may be taken:

14 cu ft of chalk weigh 1 ton	19 cu ft of gravel weigh 1 ton
18 cu ft of clay weigh 1 ton	22 cu ft of sand weigh 1 ton
21 cu ft of earth weigh 1 ton	

Rock-Excavation. A cubic yard of rock, in place, when broken up by blasting r removal by wheelbarrows or carts, will occupy a space of about 1⅔ cu yd; nsequently the cost of hauling or removal is about 50% more than for dirt.

"With labor at $1 per day, the actual cost for loosening hard rock, including ols, drilling, powder, etc., will average about 45 cents per cubic yard, in place, der all ordinary circumstances. In practice it will generally range between 30 d 60 cents, depending on the position of the strata, hardness, toughness, water d other considerations. Soft shales and other allied rocks may frequently be osened by pick and plough as low as 15 to 20 cents, while on the other hand shalw cuttings of very tough rock with an unfavorable position of strata, especially the bottoms of excavations, may cost $1 per cu yd, or even considerably more. e quarrying of average hard rock requires about ¼ to ½ lb of powder per cu yd, place, but the nature of the rock, the position of the strata, etc., may increase to ½ lb or more. Soft rock frequently requires more powder than hard. A od churn-driller will drill 8 to 10 ft in depth of holes about 2½ ft deep and 2 in ameter per day in average hard rock, at from 12 to 18 cents per ft." §

* The ordinary load for two-horse wagons such as are commonly used for hauling dirt, nd and gravel is from 1¼ to 1½ cu yd.

† Inspectors' Pocket-Book, by A. T. Byrne.

‡ See, also, Handbook of Cost Data, by H. P. Gillette.

§ The Civil Engineer's Pocket-Book, J. C. Trautwine.

DATA ON STONEWORK *

Kinds of Stonework. The commonest kind of stonework, that is, for walls is called RUBBLEWORK. No work whatever is done on the stones except to break them up with a hammer. If the wall is built in courses it is designated COURSED RUBBLE. When the stones showing on the outside face of the wall are squared, the work is designated ASHLAR. Ashlar is of two kinds: COURSED ASHLAR, in which the stones are laid to form courses around the building, all of the stones in any course being of the same height, and BROKEN ASHLAR, in which stones of different heights are used. HAMMER-DRESSED ASHLAR designates work where the stones are roughly squared with a hammer. This is a very cheap class of work. Good ashlar work should be squared on the bench with chisels, and with beds and end-joints cut square to the face. Stonework which requires a chisel or any other tool except a hammer for dressing is called CUT WORK. Cut work costs considerably more than hammer-dressed work.

Measurement of Stonework. Rough stone from the quarry is usually sold under two classifications: rubble-stone and dimension-stone. Rubble includes the pieces of irregular size most easily obtained from the quarry, and suitable for cutting into ashlar 12 in or less in height and about 2 ft long. Stone ordered to be of a certain size, to SQUARE over 24 in each way and to be of a particular thickness, is called DIMENSION-STONE. The price of the latter varies from two to four times the price of RUBBLE. Rubble is generally sold by the ton or car load. Footings and flagging are usually sold by the square foot; dimension stone by the cubic foot. In Boston, granite blocks for foundations are usually sold by the ton.

In Estimating on the Cost of Stonework put into a building, the custom varies with different localities, and even among contractors in the same city. Dimension-stone footings, that is, squared stones 2 ft or more in width, are usually measured by the square foot. If built of large rubble or irregular stones the footings are measured in with the wall, allowance being made for the projection of the footings. Rubblework is almost universally measured by the PERCH of 16½ cu ft. The author has been unable to find any locality where the legal perch of 24¾ cu ft is used by stone-masons. In Philadelphia, St. Louis and some sections of Illinois, 22 cu ft are called a perch. Railroad-work is usually measured by the cubic yard. When stonework is let by the perch, the number of cubic feet to the perch should be stated in the contract, and it should be stated also, whether or not openings are to be deducted. As a rule no deductions are made for openings of less than 70 superficial feet.

Data for Estimating Cost.† The price of common rubble as it comes from the quarry will vary from 55 cts to $1.65 per ton, free on board cars at point of delivery, according to the cost of quarrying, transportation, etc. $1.35 a perch is probably a fair average.

A ton of most of the different kinds of stones will make from 1 perch to 1½ perches.

The cost of laying one perch of stone may be estimated by the following items:

Labor: mason 2⅝ hrs, helper 1¾ hrs, based on two helpers to three masons; sand ⅛ load; lime ¾ bu, or if laid in all-cement mortar, one perch will require from ⅓ to ½ bbl of cement.

At average wages, rubble cellar-walls, from 18 in to 2 ft thick, laid in lime mortar

* The prices given are pre-war prices.

† For wages different from those named, the average costs may be calculated by proportion.

tar, vary in cost from \$2.75 to \$4.50 per perch, \$3.50 a perch being a fair average; in all-cement mortar, from \$3.50 to \$4.50 per perch.

The cost of ashlar depends very largely upon the kind of stone used and the distance it has to be brought. The price of the rough stock on the cars at the point of delivery may vary from 75 cts to \$1.35 per cu ft for granite and from 60 cts to \$1.10 for sandstones and limestones, depending largely upon cost of transportation. 1 cu ft of stone should make 2 sq ft of ashlar, at least. Some quarries get out stone especially suitable for ashlar and sell it at about 30 cts per lin ft for courses 12 in high.

The cost of cutting ashlar, with stone-cutters' wages at \$4 per day, will average about 15 cts per sq ft for soft stones, from 15 to 20 cts per sq ft for hard sandstones and limestones, and from 25 to 30 cts for granite. The cost of setting ashlar will vary from 10 cts per sq ft to 25 cts for soft stones or 30 cts for granite, 15 cts being an average price for sandstones and limestones.

The cost of cut-stone trimmings depends so largely upon the kind of stone that it is quite impossible to give prices that would be of very much service. The following figures, however, may serve as a general guide in forming a rough estimate, the prices if anything being probably a little above the cost of the local stone in most localities.

Flagstones for Sidewalks, ordinary stock, natural surface, 3 in thick, with joints pitched to line, in lengths, along walk, from 3 to 5 ft, will cost, for a 3-ft walk, about 10 cts per sq ft, or if 2 in thick, 7 cts; for a 4-ft walk, 10 cts; and for a 5-ft walk, 12 cts per sq ft. The cost of laying all sizes will average about 4 cts per sq ft. The above figures do not include the cost of hauling.

Curbing. 4 by 24-in granite will cost at the quarry from 30 to 35 cts per lin ft; digging and setting will cost from 12 to 14 cts additional; and the cost of freight and hauling must also be added.

Cut Bluestone. The following figures show the approximate cost of cut bluestone for various uses:

Flagstone, 5 in, size 8 by 10 ft, edges and top bush-hammered, per square foot face-measure	\$0.75
Flagstone, 4 in, size 5 by 5 ft, select stock, edges clean-cut, natural top, per square foot	0.45
Door-sills, 8 by 12 in, clean-cut, per linear foot	1.35
Window-sills, 5 by 12 in, clean-cut, per linear foot	0.80
Window-sills, 4 by 8 in, clean-cut, per linear foot	0.45
Window-sills, 5 by 8 in, clean-cut, per linear foot	0.60
Lintels, 4 by 10 in, clean-cut, per linear foot	0.65
Lintels, 8 by 12 in, clean-cut, per linear foot	1.25
Water-table, 8 by 12 in, clean-cut, per linear foot	1.25
Coping, 4 by 21 in, clean-cut, per linear foot	1.20
Coping, 4 by 21 in, rock-face edges and top, per linear foot	0.50
Coping, 3 by 15 in, rock-face edges and top, per linear foot	0.35
Coping, 3 by 18 in, rock-face edges and top, per linear foot	0.40
Steps, sawed stock, 7 by 14 in, per linear foot	1.10
Platform, 6 in thick, per square foot	0.50

To the prices of cut stone above given must be added the cost of setting, which, for water-tables, steps, etc., will be about 10 cts per linear foot, and for window-sills, etc., about 5 cts per linear foot. For fitting, about 10 cts per cu ft, and for trimming the joints after the pieces are set in place, about 5 cts per cu ft should also be added.

DATA ON BRICKS AND BRICKWORK

Clay Bricks. The word brick as commonly used refers to the materi clay, molded into the required shape and size and burned in a kiln; an until quite recently, practically all bricks were made from clay. At th present time, however, bricks are also made from sand and lime, and fro cement and concrete. Clay bricks may be broadly classified as commo bricks, face-bricks, fire-bricks and paving-bricks. As to the process of man facture, bricks are classified as soft-mud bricks, stiff-mud bricks, dry-press bricks and repressed bricks.

Soft-Mud Bricks are made by tempering clay with water until it becomes so and plastic and then pressing it into molds either by hand or by a machine Practically all handmade bricks are soft-mud bricks. Soft-mud bricks are ofte REPRESSED to make face-bricks.

Stiff-Mud Bricks are machine-made. The clay is first ground, and onl enough water is added to make a stiff mud. The stiff clay is forced through a di or dies in the machine in a continuous stream, which is cut up automatically int pieces the size either of the end or side of the brick. If the opening is the size o the end of the brick, the bricks are END-CUT BRICKS; if of the size of the side of th brick, they are SIDE-CUT BRICKS. Stiff-mud bricks can readily be distinguishe from soft-mud bricks by their appearance. As good if not better bricks can b made by the soft-mud process as by the stiff-mud process, and in the Easter States the soft-mud bricks are probably the stronger. As far as the author' observation has extended in the Western States, the stiff-mud bricks are as a rul preferable to those made by the soft-mud process. Stiff-mud bricks are usuall heavier than soft-mud bricks or hand-made bricks.

Dry-pressed Bricks are made almost entirely for face-work, although in some localities dry-pressed bricks are also used as common bricks. Hydraulic-pressed bricks are dry-pressed.

Molded Bricks are always dry-pressed. Very fine bricks are made by thi process.

Burning of Bricks. Bricks made by any of the above processes require to be burned in a kiln. According to their position in the kiln, common bricks ar designated ARCH-BRICKS or hard-burned bricks, RED BRICKS or well-burned bricks, and SALMON BRICKS or soft bricks. As a rule, salmon bricks are not fit to use in an exterior or bearing-wall.

Color of Bricks. The color of bricks depends principally upon the presenc of iron, lime, or magnesia in the clay. A large proportion of oxide of iron give a clear bright red. Magnesia produces a brown color, and when in the presence of iron, a light-drab color. Dry-pressed bricks are often colored artificially either by mixing clays of different composition, or by mixing mineral colors with the finely ground clay.

Fire-Bricks are ordinarily made from a mixture of flint clay and plastic clay. They are usually white, or white mixed with brown, in color and are used for th lining of furnaces, fireplaces and tall chimneys.

Paving-Bricks are very hard bricks, usually vitrified or annealed. They ar much more expensive than common bricks and are seldom used in the con struction of buildings.

Size and Weight of Clay Bricks. In this country there is no legal standard fo the SIZE OF BRICKS, and the dimensions vary with the maker and also with th

locality. Common standard sizes are 8 by 3¾ by 2¼ in. and 8 by 3⅝ by 2¼ in. In the New England States the common brick averages about 7¾ by 3¾ by 2¼ in. In most of the Western States common bricks measure about 8¼ by 4⅛ by 2⅜ in, and the thicknesses of the walls measure about 9, 13, 18 and 22 in for thicknesses of 1, 1½, 2 and 2½ bricks. The sizes of all common bricks vary considerably in each lot, according to the degree to which they are burned; the hard bricks being from ⅛ to 3/16 in smaller than the salmon bricks. In England the common standard is 8¾ by 4⅜ by 2¾ in. Pressed bricks or face-bricks are more uniform in size, as most of the manufacturers use the same size of mold. The prevailing sizes for pressed bricks are 8⅜ by 4⅛ by 2⅜ and 8⅜ by 4 by 2¼ in. Pressed bricks are also made 1½ in thick and 12 by 4 by 1½ in, those of the latter size being generally termed ROMAN BRICKS or TILES.

The WEIGHT OF BRICKS varies considerably with the quality of the clay from which they are made, and also, of course, with their size. Common bricks average about 4½ lb each, and pressed bricks vary from 5 to 5½ lb each. For the STRENGTH OF BRICKS and brickwork, see Chapter V. The FIRE-BRICKS are made in various forms to suit the required work. A straight brick measures 9 by 4½ by 2½ in and weighs about 7 lb. To secure the best results fire-bricks should be laid in the same clay from which they are manufactured, this being mixed with water into a thin paste. The thinner the joint, the better the wall will stand heat. For PAVING-BRICKS the size and weight vary according to the locality and to the requirements of the specifications. Former STANDARDS were, 2½ by 4 by 8 in, required 61 bricks to the square yard, on edge, and weighed 7 lb each. REPRESSED bricks, 2½ by 4 by 8½ in, require 58 to the square yard and weigh 6½ lb each. METROPOLITAN bricks were 3 by 4 by 9 in, required 45 to the square yard, and weighed 9½ lb each.*

Lime-Mortar Bricks.† General Description. The so-called SAND-LIME BRICKS were originally made of lime mortar, molded in brick form and hardened by exposure to the air. Such bricks are said to have been largely used in ancient times, and it is claimed that remains of such materials are now in evidence and in a good state of preservation. It is known that they were formerly used in Europe in localities where other materials were not readily available, and that they have been used in some localities in this country during the past thirty-five years. The writer knows of several houses in Haddonfield, N. J., built of such bricks, generally with the exterior surfaces plastered. One of them, however, said to be about twenty-five years old, has not been plastered, and an inspection (1915) shows the bricks to be in an excellent state of preservation. Lime-mortar bricks harden by the absorption of carbonic-acid gas from the air. This gas enters into combination with the lime, forming carbonate of lime. The hardening process requires several weeks' exposure under cover and the product has not virtues sufficient to commend it where other materials are available.

Sand-Lime Bricks. It was discovered in Germany about 1875 that lime-mortar bricks could be hardened in a few hours under heat and pressure, and it was found later that the chemical reaction under the new process differs essentially from that just described, and that the percentage of lime can be greatly reduced. The fundamental principles of sand-lime-brick manufacture are now common property and only the details of the manufacture are patentable. Sand-lime bricks were first made in Germany about 1880, and the more extended commercial development of the industry dates back in Europe to about 1888,

* Building Inspectors' Pocket-book, A. T. Byrne.

† Condensed from article on Sand-Lime Bricks by Professor Thomas Nolan in the revised edition of Building Construction and Superintendence. Part I, Masons' Work, by F. E. Kidder.

and in this country, to about 1900. There are now (1915) several factories in operation in this country.

Manufacture of Sand-Lime Bricks. Pure silica sand, mixed with from 5 to 10% of high-calcium lime and a certain proportion of water, is molded under very high pressure into the form of bricks. These are piled loosely on cars holding about 1000 bricks each and placed in a steel cylinder large enough to hold from 10 to 20 cars. The cylinder is then closed and steam is turned in and maintained at a pressure of from 120 to 135 lb to the square inch for from 8 to 10 hours, when the cylinder is opened and the bricks removed, ready for use. The tremendous pressure, which is said to be 100 tons on each brick, under which the bricks are formed, causes great density and a bringing of the component elements into close contact. The heat in the cylinder dries the bricks and causes a chemical reaction between the lime and a portion of the silica, forming a hydrosilicate of lime, an insoluble and durable element, which bonds the remaining particles of the sand together and forms a comparatively strong cementing material. The small residue of uncombined lime combines, in the course of time, either with silica or with carbonic-acid gas from the air, until no free lime remains. The bricks thus become harder and stronger with age. In regard to the constitution of sand-lime bricks, Edwin C. Eckel says:* "It may be safely assumed that a sand-lime brick as marketed consists of (1) sand-grains held together by a network of (2) hydrous lime silicate, with probably (if a magnesian lime is used) some allied magnesium silicate, and (3) lime hydrate or a mixture of lime and magnesia hydrates. These three elements will always be present, and the structural value of the brick will depend in large part on the relative percentage in which the sand and the hydrates occur."

Quality of Sand-Lime Bricks. The quality of the product depends mainly upon the selection and treatment of the sand and the lime. Pure silica sands, containing a large percentage of fine grains passing through screens of from 80 to 150 mesh, are preferable. Clay or kaolin are dangerous elements and should not be present in quantities of more than 5%. The lime should be, preferably, high-calcium lime, the magnesium silicates formed by impure limes not being as strong as calcium silicates. Some manufacturers use ready-hydrated lime, others hydrate the lime themselves, before mixing it with the sand, and others grind the quicklime, mix it with the sand and slake it in the sand. The other most important element affecting quality is the press. After pressing and before steaming, the bricks are very fragile and the press should be such that they are subjected to no shaking or friction after the pressure is removed from the mold. Vertical clay-brick presses have been commonly used, but do not appear to be well adapted to the purpose. The rotary table-presses seem to be most successful.

Tests of Sand-Lime Bricks. If the sand is reasonably clean and pure, and the lime finely divided, and if the bricks are sound and have a good metallic ring, they will stand weather-exposure well. If a brick stands in still water for an hour and the moisture rises more than 1/2 in, it is not a first-class brick; if the moisture rises 2 in, its use for facings is questionable; and if the moisture rises 3 in, it should not be used on outside work of any importance. Authentic tests† have been made for crushing, fire-resistance, frost-resistance, acid-resistance and absorption, from which it may be concluded that under proper conditions

* "The Production of Lime and Sand-lime Brick in 1906," in the Government Report dated 1907 and published in 1908, on The Mineral Resources of the United States for the Calendar Year, 1906.

† See, also, Tests Upon Sand-Lime Bricks, made by Ira H. Woolson, November, 190[illegible] at the Testing Laboratory, Columbia University, New York, for The National Association of Manufacturers of Sand-Lime Products.

manufacture sand-lime bricks are produced having the following physical characteristics: Crushing strength, average, between 2 500 and 3 000 lb per sq in, although some specimens have shown over 5 000 lb per sq in; modulus of rupture, average, about 450 lb per sq in; fire-resistance, but little inferior to that of fire-brick; frost-resistance, generally good; acid-resistance, superior; absorption, from 7 to 10% in 48 hours; rate of absorption, slower than for clay bricks; average absorption for complete saturation, 14%; reduction of compressive strength by saturation for absorption-test, average 33%.

Special Properties of Sand-Lime Bricks. The bricks are square, straight, uniform in size and homogeneous in composition and density. They cleave accurately under the stroke of the trowel and present a weather-surface with the good qualities of stone. They can be cut, carved or sand-blasted, are easily washed clean and show no efflorescence. These claims are well established for properly manufactured sand-lime bricks. It should be further stated that common bricks and facings are made in the same press, the only difference being in the selection of the materials and in the handling of the raw bricks. It is therefore claimed that a rational and homogeneous exterior wall-structure is possible, since backings and facings may be built and bonded in even courses, with Flemish or other ornamental bonds. Some factories, however, manufactured, at first, inferior bricks and care should still be taken in selections from their outputs. Frequently the ordinary runs of sand-lime bricks are not as strong as the average clay building bricks and some of them are too low in their resistance to frost.

Colors of Sand-Lime Bricks. The natural color is pearl-gray, varying in warmth with the composition of the sand. Permanent colors are produced by introducing mineral oxides with the raw materials in quantities varying according to the intensity of color desired; but as the oxides are foreign materials in the bricks, they affect the quality of the latter in proportion to the quantity used.

Glazed and Enameled Bricks. The term GLAZED BRICKS and ENAMELED BRICKS as commonly used, refer practically to the same product, and neither includes what is known as SALT-GLAZED BRICK. The enameled or glazed bricks are generally dipped or sprayed and then burned, whereas the salt-glaze is obtained by the introduction of salt into the fire-boxes of kilns while the bricks are being burned. Glazed or enameled bricks are generally divided into two classes: (1) true enameled bricks, which have a glaze containing the coloring matter applied to it without any intermediate SLIP; (2) bricks which have a transparent glaze placed over a white or colored slip, the slip coming between the glaze and the material to be glazed. The latter is the process most used in this country. Manufacturers differ as to which process produces the best bricks although it would seem as though the true enamel would not chip or peel as readily. These bricks can be made in a variety of colors, from white to dark green or chocolate, and either in a HIGHLY GLAZED FINISH or in a DULL, SATIN-FINISH, the latter finish being quite desirable in many instances on account of its doing away with the glare of the more highly glazed bricks or tiles. An enameled surface may be distinguished from a glazed surface by chipping off a piece of the brick. The glazed brick will show a layer of slip between the glaze and the body of the brick; while the enameled brick will show no line of demarcation between the body of the brick and the enamel. American enameled and glazed bricks are now extensively used for the exterior surfaces of buildings, particularly for street-fronts and light-courts, and for interior side walls and partitions of rooms or buildings used for a great variety of purposes.

Sizes of Enameled Bricks. Enameled bricks are made in two regular sizes: (1) English size, 9 by 3-in enameled surface, 4½-in bed, and (2) American size, 8⅜ by 2¼-in enameled surface, 4⅛-in bed. The English-size bricks cost about $10 per 1 000 more than the American, but on account of the saving in the number of bricks, labor of laying and mortar in joints, the former really effect a saving of about 7 cts per sq ft. Enameled bricks are made, also, with a 12 by 4½-in enameled surface, 2¼-in bed.

Cost of Enameled Bricks.* The selling price of enameled bricks varies from $75 per 1 000 for the American size to $85 for the English size and $100 for the 12 by 4½ by 2¼-in size; and at these prices the cost of the bricks per square foot is:

	cts
American size, 7 bricks to the foot	52½
English size, 5⅓ bricks to the foot	45⅓
English flat, 3⅗ bricks to the foot	36
12 by 4½ by 2¼-in, 3 bricks to the foot	30

Colors of Enameled Bricks. The standard colors carried in stock are white, cream and buff; other colors are made to order.

Estimating Quantities and Cost of Brickwork *

Methods of Calculation. The almost universal method of calculating the cost of brickwork is by estimating the number of thousands of bricks, WALL MEASURE, and then multiplying by a certain price per thousand, which is usually determined by experience and which is intended to include every item affecting the cost, and very often the profit. All of the common brickwork in any given building is usually figured at the same price per thousand bricks, the adjustment for the more expensive portions of the work being made in the manner of measuring. The principle underlying this system is explained as follows:

"The plain dead wall of brickwork is taken as the standard, and the more difficult, complicated, ornamental, or hazardous kinds of work are measured up to it so as to make the compensation equal. To illustrate, if, in one day, a man can lay 2 000 bricks in a plain dead wall, and can lay only 500 in a pier, arch, or chimney-top in the same time, the cost of labor per thousand in such work is four times as much as in the dead wall, and he is entitled to extra compensation; but instead of varying the price, the custom is to vary the measurement to compensate for the difference in the time, and thus endeavor to secure a uniform price per thousand for all descriptions of ordinary brickwork, instead of a different price for the execution of the various kinds of work." †

Measurements of Brick-Quantities. PLAIN WALLS are quite universally figured at 15 bricks to the square foot of an 8 or 9-in wall, 22½ bricks per square foot of a 12 or 13-in wall, 30 bricks per square foot of a 16 or 17-in wall, and 7½ bricks for each additional 4 or 4½ in in the thickness of the wall. These figures are used without regard to the size of the bricks, the effect of the latter being taken into account in fixing the price per thousand. No deduction is made for OPENINGS of less than 80 sup ft, and when deductions are made for larger openings the width is measured 2 ft less than the actual width. HOLLOW WALLS are all measured as if solid. To the number of bricks thus obtained is added

* The prices given are pre-war prices.

† From Rules of Measurement adopted by the Brick Contractors' Exchange of Denver, Col.

measurement for piers, chimneys, arches, etc. FOOTINGS are generally measured in with the wall by adding the width of the projection to the height of the wall. Thus if the footings project 6 in on each side of the wall, 1 ft is added to the actual height of the wall. CHIMNEY-BREASTS and PILASTERS are measured by multiplying the girth of each breast or pilaster from the intersections with the wall by the height, and then by the number of bricks corresponding with the thickness of the projection. FLUES in chimneys are always measured solid. Detached CHIMNEYS and CHIMNEY-TOPS are measured as a wall having a length equal to the sum of the side and two ends of the chimney, and a thickness equal to the width of the chimney. Thus a chimney measuring 3 ft by 1 ft 4 in would be measured as a 16 or 17-in wall, 5 ft 8 in long. The rule for INDEPENDENT PIERS is to multiply the height of each pier by the distance around it in feet, and consider the product as the superficial area of a wall whose thickness is equal to the width of the pier. In practice, many masons measure only one side and one end of a pier or chimney. ARCHES of common bricks over openings of less than 80 sup ft are usually disregarded in estimating. If the arch is over an opening larger than 80 sq ft, the height of the wall is measured from the springing-line of the arch. No deduction is made in the wall-measurement for stone sills, caps, or belt-courses, nor for stone ashlar, if the same is set by the brick-mason. If the ashlar is set by the stone-mason, the thickness of the ashlar is deducted from the thickness of the wall. The sum of all of these measurements represents a certain number of THOUSANDS OF BRICKS, and the whole is then multiplied by a common price per thousand, as \$6, \$8, \$12, or \$16, according to whatever the cost of plain brickwork may be. If the building is to be faced with PRESSED BRICKS, the actual cost of the pressed bricks, as nearly as it can be computed, is added to the estimated price of the common brickwork, nothing being added for laying the pressed bricks, nor anything deducted from the common-brick measurement, the measurement of the common work displaced by the pressed bricks being assumed to offset the difference in the cost of laying the pressed and common brickwork. In arriving at the COST OF THE PRESSED BRICKS, the external superficial area of the walls faced with such bricks is computed, and all openings, belt-courses, stone caps, etc., are deducted. Five-in stone sills are not usually deducted. If a portion of the wall is covered by a porch, so that common bricks may be used back of it, this space, also, is deducted. The net pressed-brick surface is then multiplied by 6, 6½, or 7 to obtain the number of bricks required, 6½ giving about the number of pressed bricks of the standard size required to the square foot. The TOPPING OUT of chimneys, if of face-brick, is measured by girting the chimneys, multiplying by the heights, and adding the sums to the wall-area.

Example. As a simple example of this system of estimating consider a small brick house, 28 by 32 ft in plan, without cross-walls, the basement-walls being in thick, with footings 2 ft 6 in wide; the first-story walls, 13 in thick; the second-story walls, 9 in thick; the height of the basement-walls from the trench to the top of the first-story joists, 8 ft 6 in; the height of the walls from the first-story joists to the top of the second-story joists, 10 ft 6 in; and from the second-story joists to the plate, 9 ft.

WALL-MEASUREMENTS. Basement-walls: 120 ft (girth of building) by 9 ft in (height and projection of footing) by 22½ bricks per square foot; equal to 550 bricks.

First-story walls: 120 ft by 10 ft 6 in by 22½ bricks per square foot; equal 28 360 bricks.

Second-story walls: 120 ft by 9 ft by 15 bricks per square foot; equal to 200 bricks.

Topping out two chimneys, each 1 ft 9 in by 1 ft 5 in by 14 ft high above roof:

2 by 14 ft by (1 ft 5 in plus 1 ft 9 in plus 1 ft 5 in) by 30 bricks per square foot; equal to 3 850 bricks.

Total brickwork: 74 960 bricks. At $9 per 1 000, the cost is $674.64.

PRESSED BRICKS. From the grade to the under side of the plates, the wall measures 22 ft 6 in and it is to be faced with pressed bricks of the standard size, costing $15 per 1 000. The door-openings and window-openings measure 384 sup ft.

The surface of pressed bricks equals 120 by 22½ ft, equal to.....	2 700 sq ft
The deduction for openings is..............................	384 sq ft
Area, after deduction..................................	2 316 sq ft
Addition for two chimneys, 2 by 14 by 6 ft 4 in, equal to.......	177 sq ft
Total	2 493 sq ft

2 493 by 6½ equals 16 204 pressed bricks, which, at $15 per 1 000 cost, equals $243.

The total amount of the bid is $674.64 plus $243, or $917.64.

The above figures are supposed to include the necessary lime, sand, water, scaffolding, etc., required to make the mortar and put up the walls, and also a profit for the contractor; but anything in the way of ironwork, such as ties, thimbles, ash-doors, etc., are figured as additions to this amount.

Detailed Estimates of Brickwork. In estimating by the above method, the price per thousand is to some extent a matter of guesswork, and while an experienced contractor may perhaps make as accurate an estimate by this method as is possible by any, yet it is often necessary to estimate the work in detail; and even when the work has been estimated as above, it is necessary for the contractor to know how many bricks and how much sand and lime will be required to do the work. The following data will assist in making such detailed estimates.

With the size of bricks used in the Western States, from 16½ to 17⅔ common bricks are required to the cubic foot after deducting openings, and figuring the thickness of walls at 8, 12, 16, 20 in, etc., the actual number of bricks required will run about two-thirds of the WALL-MEASURE when the openings are of about the average number and size.

The number of pressed bricks will be about 6 or 6½ bricks to the foot, after deducting openings.

To lay 1 000 common bricks, kiln-count, requires 2½ bushels or 200 lb of white lime and ⅝ cu yd of sand. For a good lime-and-cement mortar, allow 2 bushels of lime, 1 bbl of cement and ⅝ cu yd of sand. For 1 : 3 cement-and-sand mortar, allow 1½ bbl of cement and ⅝ cu yd of sand, or one-half a load.

To lay 1 000 pressed bricks with buttered joints will require 2 bushels of lime (160 lb) and ¼ cu yd of sand; with spread joints, from 2 to 2½ bushels of lime and from ⅜ to ½ cu yd of sand.

If colored mortar is used, about $1 per 1 000 bricks should be added for the mortar-color.

A brick-mason, working on a city job under a good foreman, will lay, on an average, 60 pressed (face) bricks per hour, and from 150 to 175 common bricks per hour, 160 being a fair average. In country towns the average is nearer [illegible] per hour.

With wages at 62½ cts per hour for masons, 31¼ cts for hod-carriers, and 3[illegible] cts for mortar-mixers and carriers, sand at 60 cts per cu yd, and lime at 40 cts per bushel of 80 lb, brick-masons in Denver state that the average cost of laying common bricks in 12-in walls is about $6 per 1 000, kiln-count, and of laying pressed bricks about $10 per 1 000.

For common brickwork, one helper will be required for every mason, and on 9-in walls, faced with pressed bricks, one helper to every two masons. In building common-brick fireplaces and chimneys one mason and helper will lay about 600 bricks in a day of nine hours.

As a rule, chimneys built of common bricks and with 4-in walls cost about 50 cts per running foot, in height, for single flues, and 90 cts for double flues.

Space Required for Piling Bricks. One thousand bricks closely stacked occupy about 56 cu ft of space. One thousand old bricks, cleaned and loosely stacked, occupy about 72 cu ft.

A brick-layer's hod measures 21 by 7 by 7 in, and will hold 18 bricks.

A mortar-hod measures 24 by 12 by 12, and 12 in across the top.

Mortar-Colors are usually in the form of dry powders, or of pulp or paste. The powders are put up in barrels, the number of pounds to the barrel and price per pound being about as follows:

Red, in 500-lb barrels, dry	from 1¾ to 2 cts per lb
Brown, in 450-lb barrels, dry	from 1¾ to 2½ cts per lb
Buff, in 400-lb barrels, dry	from 1¾ to 2½ cts per lb
Black, in 1 000-lb barrels, dry	from 3 to 3½ cts per lb

For lots of less than full barrels an extra charge is sometimes made for packing and drayage.

In pulp or paste-form:

Red, brown and buff	1¾ cts per lb
Black	3 cts per lb
All other colors	2 cts per lb

Colors in paste-form can be obtained in casks, barrels, half-barrels and kegs, all (except black and buff) weighing, in casks, 900 lb; in barrels, 550 lb; and in half-barrels, 375 lb. The buff weighs, in casks, 700 lb; in barrels, 450 lb; and in half-barrels, 300 lb. Black weighs, in barrels, 450 lb; and in half-barrels, 275 lb. To color the mortar for laying 1 000 bricks with ⅛-in joints requires about 50 lb of red, terra-cotta color, amber, fern-green and salmon; 40 lb for buff, brown, colonial drab or French gray; and 25 lb for black. For wider joints, a larger quantity of stain must be used. For paste-colors an average mixture is, 1 bucket of paste-color to 7 buckets of mortar for brickwork with ⅛-in joints. When the colors are in the form of dry powder they are first mixed with dry sand, the cold slaked lime is then added and again mixed thoroughly. It is very important that the color be uniformly mixed. If it is not added at first, but left until the mortar is made, the labor of mixing is doubled. The more thorough the mixing the less color is required. Mortar colors should never be mixed with hot lime. When the color is in the form of a pulp or paste, it should be thoroughly hoed in, in order to secure a uniform and smooth shade. For very fine pressed bricks, the stained mortar should be strained through a coarse sieve.

Efflorescence on Brickwork. A white EFFLORESCENCE often appears on brickwork, especially in moist climates and damp places. It may spread over large areas of the wall-surface although originating in the mortar joints. Soluble salts, principally of soda, potash and magnesia, in the cement or lime mortar, are dissolved by the water absorbed by the mortar and later precipitated on the surface of the brickwork as a white deposit, when the water evaporates. This deposit seems to be greater with the natural than with the Portland-cement mortars and still heavier with lime mortar. The origin of the efflorescence may be in the bricks themselves as well as in the mortar used. This is the case when the bricks are made from clays containing iron pyrites or burned with sulphurous

coal. Moisture in such bricks tends to dissolve the sulphate of magnesia and sulphate of lime, which, in the evaporation of the water, are deposited on the surface as crystals of these salts. Efflorescence may result, also, from water impregnated from the mortar, absorbed by the bricks and then evaporated, leaving the whitish deposits; and it is sometimes caused by adulterations in certain MORTAR-COLORS. As a PREVENTIVE, General Gilmore recommended the addition to every 300 lb of the cement powder, 100 lb of quicklime, and from 8 to 12 lb of any cheap ANIMAL FAT, which is to be thoroughly incorporated with the quicklime before the latter is slaked, preparatory to adding it to the cement. The alkaline salts tend to be SAPONIFIED by the fat. This is not an entirely satisfactory treatment, and as a rule it only partly prevents or removes the objectionable deposits; and this addition to the cement retards its setting and somewhat diminishes its strength. It is claimed by some that boiled LINSEED-OIL, applied to brickwork in two coats, will lessen the absorption of moisture for from one to three years and thus lessen the tendency to efflorescence. It is usually mixed in the proportion of 2 gal of oil to 300 lb of dry cement, either with or without lime; but it is injured by the mortar and, like the fat, retards the setting of the cement mortar and weakens it. In order to diminish the chances of efflorescence on brickwork, the walls should be made as IMPERVIOUS as possible by laying the bricks in a rich well-mixed Portland-cement mortar and filling all joints full and solid. If the building is on damp ground, carefully constructed DAMP-PROOF COURSES of the proper materials should be built into the walls or a course of horizontal joints near the bottom of the walls should be WATERPROOFED. Reasonably hard bricks should be used for facing, projections and exposed top surfaces waterproofed and provided with drips, and the roof, cornice and gutters made water-tight. When efflorescence is due to the penetration of rain-water or moisture into the brickwork and it is required to preserve the texture and color of the work, the surface may be coated with preparations of PARAFFINE or with various patented WATERPROOFING MIXTURES. The preparations containing paraffine are usually applied hot, and the walls, also, are heated by portable heaters previous to the application. They give fairly good results, but are quite expensive, owing to the time and labor required for their application. Brick walls may be rendered impervious to moisture by washes applied by the SYLVESTER PROCESS. These washes consist of an ALUM-SOLUTION made by dissolving 1 lb of alum per gallon of water, and a SOAP-SOLUTION made by dissolving 2½ lb of pure hard soap per gallon of water. The brick walls should be dry and clean and it is recommended that they should not be colder than 50° F. The soap-wash is made boiling hot and then applied to the brickwork. The temperature of the alum-solution is usually from 60° to 70° F. when put on. One wash is applied and allowed to dry for about 24 hours, after which the other wash is put over it. When ALUMINIUM SULPHATE, improperly called ALUM, is substituted for the alum, the cost of the wash is less, only two-thirds as much sulphate as alum is required and the results are better.

LIME *

Nature and Properties of Lime. Chemically, lime is calcium oxide. Used in a broader sense, it is the class-name of a great variety of products manufactured by the calcination of LIMESTONE. Limestone consists of the carbonates of calcium and magnesium which vary widely in their ratio to each other. The limestones used in the manufacture of lime products may be divided into two

* Valuable practical data relating to lime and plaster has been furnished by the Charles Warner Company, of Wilmington, Del.

classes, CALCIUM LIMESTONES and DOLOMITIC LIMESTONES. High-calcium limestones contain only a relatively low percentage of magnesium carbonate, while dolomitic limestones contain a considerable amount of it. Dolomitic limestone usually corresponds roughly to the theoretical formula of dolomite ($CaCO_3$) ($MgCO_3$). The CALCINATION of limestone consists of heating to expel the carbon dioxide. The product resulting from calcination of limestone is known as QUICKLIME and possesses great affinity for water. SLAKING is the process of adding water to quicklime. During the process of slaking, heat is energetically evolved and much of the water driven off in the form of steam. During this slaking process, also, high-calcium quicklimes must be agitated and stirred continually or a portion will fail to receive the proper quantity of water and will contain unslaked particles which are likely to slake after being used in the work, causing POPPING, PITTING and disintegration. Dolomitic limes do not slake so energetically, and while they should be stirred while slaking, this is not so necessary as with high-calcium limes. Either class of quicklime, through faulty manufacture, is likely to contain over-burned portions which slake with difficulty and may cause popping, etc., if the lime-paste is not carefully screened before use. The SETTING and HARDENING of common lime mortar is due, first, to the drying out and, secondly, to the absorption of carbon dioxide from the atmosphere and the formation of crystals of calcium carbonate to which the strength of the mortar is ascribed. In the manufacture and use of common lime mortar, therefore, the raw material, limestone, is first calcined, and the carbon dioxide expelled; it is then slaked with water and forms calcium hydroxide, in which the water is gradually replaced by carbon dioxide. The lime thus eventually returns to its original carbonate form. As far as the ultimate result is concerned, there is generally little difference between high-calcium and dolomitic quicklimes. Owing to greater familiarity with one or the other of the classes of lime, architects and builders in certain sections of the country prefer one to the other.

Specifications for Quicklime. The lime industry has in recent years been made the subject of careful study and the following clauses give the various requirements of Standard Specifications for Quicklime adopted by the American Society for Testing Materials in 1915.

1. DEFINITION. Quicklime is a material the major part of which is calcium oxide or calcium and magnesium oxides, which will slake on the addition of water.

2. GRADES. Quicklime is divided into two grades:

(*a*) Selected. Shall be well-burned, picked free from ashes, core, clinker or other foreign material.

(*b*) Run-of-Kiln. Shall be well-burned, without selection.

3. FORMS. Quicklime is shipped in two forms:

(*a*) Lump. Shall be kiln-size.

(*b*) Pulverized Lime. Lump lime reduced in size to pass a ¼-in screen.

4. CLASSES. Quicklime is divided into four classes: (*a*) High-Calcium; (*b*) Calcium; (*c*) Magnesian; (*d*) High-Magnesian.

5. BASIS OF PURCHASE. The particular grade, form and class of quicklime desired shall be specified in advance by the purchaser.

I. Chemical Properties and Tests

(A) Sampling

6. LIME IN BULK. When quicklime is shipped in bulk, the sample shall be so taken that it will represent an average of all parts of the shipment from top to bottom, and shall not contain a disproportionate share of the top and bottom layers, which are most subject to changes. The samples shall comprise at least shovelfuls taken from different parts of the shipment. The total sample

taken shall weigh at least 100 lb and shall be crushed to pass a 1-in ring and quartered to provide a 15-lb sample for the laboratory.

7. LIME IN BARRELS. When quicklime is shipped in barrels, at least 3% of the number of barrels shall be sampled. They shall be taken from various parts of the shipment, dumped, mixed and sampled as specified in Section 6.

8. LABORATORY SAMPLES. All samples to be sent to the laboratory shall be immediately transferred to an air-tight container in which the unused portion shall be stored till the quicklime is finally accepted or rejected by purchaser.

(B) Chemical Tests

9. CHEMICAL PROPERTIES. (*a*) The classes and chemical properties of quicklime shall be determined by standard methods of chemical analysis. (*b*) Samples shall be taken as specified in Sections 6, 7 and 8. (*c*) Quicklime shall conform to the following requirements as to chemical composition:

CHEMICAL COMPOSITION

Properties considered	High-Calcium		Calcium		Magnesian		High-Magnesian	
	Selected	Run of kiln	Selected	Run of kiln	Selected	Run of kiln	Selected	Run of kiln
Calcium oxide, per cent..	90 (min)	90 (min)	85–90	85–90	..	..	..	..
Magnesium oxide, per ct.	..	..	..	..	10–25	10–25	25 (min)	25 (min)
Calcium oxide plus magnesium oxide, min, per cent................	90	85	90	85	90	85	90	85
Carbon dioxide, max, per cent................	3	5	3	5	3	5	3	5
Silica plus alumina plus oxide of iron, max, per cent...............	5	7.5	5	7.5	5	7.5	5	7.5

II. Physical Properties and Tests

10. PERCENTAGE OF WASTE. An average 5-lb sample shall be put into a box and slaked by an experienced operator with sufficient water to produce the maximum quantity of lime putty, care being taken to avoid burning or drowning the lime. It shall be allowed to stand for 24 hours and then washed through a 20-mesh sieve by a stream of water having a moderate pressure. No material shall be rubbed through the screens. Not over 3% of the weight of the selected quicklime nor over 5% of the weight of the run-of-kiln quicklime shall be retained on the sieve. The sample of lump lime taken for this test shall be broken so that all of it will pass a 1-in screen and be retained on a ¼-in screen. Pulverized lime shall be tested as received.

III. Inspection and Rejection

11. INSPECTION. (*a*) All quicklime shall be subject to inspection.

(*b*) The quicklime may be inspected either at the place of manufacture or the point of delivery, as arranged at time of purchase.

(*c*) The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the quicklime ordered. The manufacturer shall afford the inspector all reasonable facilities for inspection and sampling, which shall be so conducted as not to interfere unnecessarily with the operation of the works.

(*d*) The purchaser may make the tests to govern the acceptance or rejection of the quicklime in his own laboratory or elsewhere. Such tests, however, shall be made at the expense of the purchaser.

12. REJECTION. Unless otherwise specified, any rejection based on failure to pass tests prescribed in accordance with these specifications shall be reported within five days from the taking of samples.

13. REHEARING. Samples which represent rejected quicklime, shall be preserved in air-tight containers for five days from the date of the test-report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

Hydrated Lime. The slaking of quicklime is an operation which is almost invariably carried on by laborers who have little or no conception of the importance of their task. As a result, many failures have been charged to lime in the past which actually were due to improper preparation during the slaking operation. The new product known as HYDRATED LIME has been offered widely to the trade in recent years and has met with much success. Hydrated lime is a dry flocculent powder resulting from the slaking of quicklime by mechanical means, with an amount of water which is sufficient to satisfy the calcium oxide, but insufficient to make a paste or putty. Hydrated lime is manufactured in mechanical hydrators in which the batches of quicklime and water used are carefully proportioned by weight. After passing from the hydrator, hydrated lime is subjected to a mechanical system of separation which eliminates the coarse or impure particles which may cause popping, etc. Hydrated lime is sold in bags of definite weight and requires only to be mixed with sand and water to make the mortar. The bags have usually been made of heavy burlap or duck cloth, containing 100 lb, or of paper, containing 40 lb. Several of the more prominent manufacturers of hydrated lime in the United States employ chemists who regularly superintend the manufacture of hydrated lime, just as the chemists in Portland-cement factories superintend the proportioning of the raw mix going to the kilns to be burned for Portland cement. The hydrated lime manufactured under such chemical supervision is a reliable product free from tendencies which might give rise to popping, pitting or disintegration. Hydrated lime of good quality may be used for almost any purpose for which lime mortar is used, and is by some considered a more reliable product than quicklime. Among the newer uses for hydrated lime may be mentioned its employment in cement mortars and concrete. An addition of about 15% of hydrated lime to cement mortar or concrete decreases its permeability to water, reduces the cracking due to shrinkage, etc., and increases the plasticity of the mortar or concrete, thus preventing separation of the sand, stone and cement and causing the mixture to flow and fill the forms more readily. (See Macgregor tests, page 276.)

Specifications for Hydrated Lime. The following clauses give the various requirements of Standard Specifications for Hydrated Lime adopted by the American Society for Testing Materials in 1915.

1. DEFINITION. Hydrated lime is a dry flocculent powder resulting from the hydration of quicklime.

2. CLASSES. Hydrated lime is commercially divided into four classes: (*a*) High-Calcium; (*b*) Calcium; (*c*) Magnesian; (*d*) High-Magnesian.

3. BASIS OF PURCHASE. The particular type of hydrated lime desired shall be specified in advance of purchase.

I. Chemical Properties and Tests

4. SAMPLING. The sample shall be a fair average of the shipment. Three per cent of the packages shall be sampled. The sample shall be taken from the surface to the center of the package. A 2-lb sample to be sent to the laboratory

shall immediately be transferred to an air-tight container, in which the unused portion shall be stored until the hydrated lime has been finally accepted or rejected by the purchaser.

5. Chemical Properties. (*a*) The classes and chemical properties of hydrated lime shall be determined by standard methods of chemical analysis. (*b*) The non-volatile portion of hydrated lime shall conform to the following requirements as to chemical composition:

Chemical Composition

Properties considered	High-Calcium	Calcium	Magnesian	High-Magnesian
Calcium oxide, per cent.....	90 (min)	85–90	....	....
Magnesium oxide, per cent..	....	....	10–25	25 (min)
Silica plus alumina plus oxide of iron, max, per cent	5	5	5	5
Carbon dioxide, max, per cent	5	5	5	5
Water....................	Sufficient to hydrate the calcium-oxide content	Sufficient to hydrate the calcium-oxide content	Sufficient to hydrate the calcium-oxide content	Sufficient to hydrate the calcium-oxide content

II. Physical Properties and Tests

6. Fineness. A 100-g. sample shall leave by weight a residue of not over 5% on a standard 100-mesh sieve and not over 0.5% on a standard 30-mesh sieve.

7. Constancy of Volume. Hydrated lime shall be tested to determine its constancy of volume in the following manner: Equal parts of hydrated lime under test and volume-constant Portland cement shall be thoroughly mixed together and gauged with water to a paste. Only sufficient water shall be used to make the mixture workable. From this paste a pat about 3 in in diameter and ½ in thick at the center, tapering to a thin edge, shall be made on a clean glass plate about 4 in square. This pat shall be allowed to harden 24 hours in moist air and shall be without popping, checking, cracking, warping or disintegration after 5 hours' exposure to steam above boiling water in a loosely closed vessel.

III. Packing and Marking

8. Packing. Hydrated lime shall be packed either in cloth or paper bags and the weight shall be plainly marked on each package.

9. Marking. The name of the manufacturer shall be legibly marked or tagged on each package.

IV. Inspection and Rejection

10. Inspection. (*a*) All hydrated lime shall be subject to inspection.

(*b*) The hydrated lime may be inspected either at the place of manufacture or the point of delivery, as arranged at the time of purchase.

(*c*) The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the hydrated lime ordered. The manufacturer shall afford the inspector all reasonable facilities for inspection and sampling, which shall be so conducted as not to interfere unnecessarily with the operation of the works.

(*d*) The purchaser may make the tests to govern the acceptance or rejection of the hydrated lime in his own laboratory or elsewhere. Such tests, however, shall be made at the expense of the purchaser.

11. Rejection. Unless otherwise specified, any rejection based on failure to pass tests prescribed in these specifications shall be reported within five working days from the taking of samples.

12. Rehearing. Samples which represent rejected hydrated lime shall be preserved in air-tight containers for five days from the date of the test-report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

Alca Lime. A recent development in the lime industry is Alca Lime.* This is a material said to combine the plasticity and sand-carrying qualities of lime mortar with the strength, hardness and quicker set of the gypsum plasters. It is composed of approximately 85% of hydrated lime and 15% of a specially prepared material containing alumina and silica in such proportions as to combine, forming bodies which greatly contribute to the strength, hardness and plasticity of the product. It is sold in 100-lb packages and requires only to be mixed with sand and water before use. When used for plastering, it has the characteristics of lime mortar, and while it becomes hard and strong, it is claimed that it is free from the so-called sounding-board effects noticed in some hard-wall plasters. It is not injured by water and is often used for outside stucco-work and also as a brick-laying mortar in place of lime mortar gauged with Portland cement. The manufacturers' directions for the use of Alca Lime should be carefully observed, and this may be said of all prepared plastering or cementing materials.

Useful Data on Quicklime. Quicklime is shipped either in barrels or in bulk. In dry climates it will keep for a long time in bulk, but in damp climates and along the coast it soon slakes unless enclosed in barrels. By Act of Congress, August 23, 1916, it is required that lime in barrels shall be packed only in barrels containing 280 lb or 180 lb, net weight. When shipped in bulk it is generally sold by the bushel of 80 lb, 3½ bushels or 280 lb, net, of lime being considered as equivalent to a large barrel. Other weights are 180 lb, net, per small barrel, and 64 lb per cu ft. The average yield of LIME-PASTE from the best Eastern limes has been found to be 2.62 times the bulk of unslaked lime. A barrel of good quality well-burned lime should make 8 cu ft, or 20 pails, of lime-paste or putty. Careful experiments conducted by United States engineers have demonstrated that the best mortar is obtained by mixing one part of lime paste to two parts of sand.

Cements. For data on cements, see Chapter III.

SAND AND GRAVEL

Sand is obtained from banks or pits, from river-beds and from the seashore. Pit-sand or bank-sand, free from clay or earthy materials, is generally considered the best for mortar, although excellent sand is often obtained from river-beds. Sea-sand contains alkaline salts which attract and retain moisture and which, unless thoroughly washed, cause efflorescence when used in brickwork. Both sea-sand and river-sand have more or less rounded grains, to which lime or cement will not adhere as well as to sharp, angular grains. Both are extensively used, however, for lack of better materials. The use of sand in mortar is to prevent excessive shrinkage and to save the cost of lime or cement. Sand, when used in the proportion of 1 : 2, strengthens lime mortar, but any addition of sand to cement weakens it.

Screening Sand. Sand for mortar must ordinarily be screened. Sand for brown mortar for plastering or common brickwork is ordinarily run through a

* This is a patented article and is offered for sale by many licensees in the United States under the Spackman patents.

No. 4 screen having 4 by 4 meshes to the inch. For sand finish and mortar pressed brickwork, either a No. 10 or a No. 12 screen with 10 by 10 or 12 by meshes to the inch is commonly used. For rubble stonework the sand is ordinarily screened, unless it contains much gravel, in which case it should screened through a ⅜-in mesh.

Weight of Sand. Dry sand weighs from 80 to 115 lb per cu ft. The average weight of damp (not wet) sand is about 96 lb per cu ft, or about 2 600 per cu yd. The voids for ordinary sand range from 0.3 to 0.5 of the volume, the average for screened sand suitable for mortar being 0.35 of the volume. The more uneven the grains in size the smaller the percentage of the voids. A one-horse load of sand contains about 22 cu ft. Two-horse loads vary from 1¼ to 2 yd. The amount hauled per load in the larger cities is generally fixed by the Team Owners' Association. 1¼ yd is a fair load, 1½ yd a good load and 2 yd a large load.

LATHING AND PLASTERING

Wooden Laths should be well seasoned, free from sap, bark and dead knots. Bark on laths is quite sure to stain the plaster. White pine is generally considered the best wood for laths, although spruce and hemlock laths are much used. Hard pine is not a good material, as it contains too much pitch. The regular size of laths is ¼ in by 1½ in by 4 ft. The width and thickness vary somewhat in different mills. There is a new lath on the market, which is only 32 in long and which costs from $1.75 to $2 less than the 48-in lengths. Laths are sold by the thousand, in bunches containing 100 laths, from $4.50 to $5.50 being about the average prices. (Pre-war prices.)

Metal Lathing. (See Chapter XXIII, pages 883 to 887.)

Plastering on laths is generally done in three coats.* The first coat is called the SCRATCH-COAT; the second, the BROWN COAT, and the third, the WHITE COAT, SKIM-COAT, or FINISH. On brickwork or stonework the scratch-coat is generally omitted. For first-class work each coat should be permitted to dry thoroughly before the next coat is applied, and under no circumstances should the finish-coat be applied before the brown coat is thoroughly dry.

Drawn Work is a brown coat applied to a scratch-coat from the same staging, immediately after the scratch-coat is applied. It is a little cheaper than DRY SCRATCH, and much of it is done in the Western States.

The Scratch-Coat should always be made rich in lime, and should contain 1¼ bu of hair, or an equivalent quantity of fiber to each cask of lime, or 1 bu of hair to 2 of lime. A proportion of one part lime-paste to two parts of sand will require 1 cask (2½ bu) of lime to 5½ bbl of screened sand.

The Brown Coat should contain 1 cask (2½ bu) of lime to 7 bbl of screened sand, and 1 bu of hair to 5 of lime. Very little plaster is mixed by measure, however, the usual custom being to mix as much sand with the slaked lime as the mortar-mixer thinks it will stand and give satisfaction, the tendency being always to make the lime go as far as possible.

The Third or Finishing Coat is designated by various terms, such as SKIM-COAT, WHITE COAT, PUTTY-COAT, SAND-FINISH, etc. The skim-coat as used in the

* In the Eastern States, dwellings of moderate cost are generally plastered with two-coat work, the first or scratch-coat being brought nearly to the grounds, and carefully [illegible]ed to receive the skim-coat.

Eastern States is generally composed of lime-putty and washed beach-sand in equal proportions.

Sand Finish, which has a rough surface resembling coarse sandpaper, is mixed in the same way, only that coarser sand and more of it is used, and it is finished with a wooden or cork-faced float.

White Coating or Hard Finish generally means a composition of lime-putty and plaster of Paris, to which marble-dust is sometimes added. Plaster of Paris and marble-dust when used should not be mixed with the lime-putty until a few moments before using, and no more should be prepared at one time than can be used up at once, as it soon SETS, after which it should not be used. The skim-coat or hard finish should be finished with a steel trowel and wet brush. The more the work is troweled the harder it becomes. A superior hard finish is obtained by mixing 4 parts of Best's Keene's cement to 1 part lime-putty.

Mortar for Plastering. To make sure that the lime is well slaked, it is customary to require that the mortar for plastering shall be mixed at least seven days before it is used.

Hair such as is used by plasterers is obtained from the hides of cattle, and after being washed and dried is put up in paper bags, each bag being supposed to contain 1 bushel of hair when beaten up. Each package is supposed to weigh from to 8 lb but the weight often falls short. ASBESTOS and MANILLA FIBER are both used in place of hair; they are cleaner than hair and are said to be less injured by the lime. It is much better to add the hair to the lime-paste AFTER IT IS COLD and before mixing in the sand, as hot lime, and the steam caused by the slaking, burn or rot the hair so as to greatly weaken it. The common practice is to put the hair in the mortar-box, run off the hot lime as soon as it is slaked, throw in the sand and mix the whole together. It is then thrown out of the box into a pile and a new batch mixed up.

Machine-Made Mortar. In several of the larger cities plants have been equipped for the mixing of mortar by machinery. Machine-mixed mortar should be much better than the ordinary hand-mixed mortar, for the reason that time can be given for the lime to slake, the lime and sand can be accurately measured, and the hair and lime are not mixed with the lime until just before delivery. The mixing may also be more thoroughly and evenly done by machinery than is possible by hand.

Improved Wall-Plasters. Owing to the difficulty of obtaining sufficient space in building operations in central sections of large cities to properly slake sufficient lime mortar to carry on the plastering with the necessary speed, other kinds of plastering materials have come into existence in recent years. These are known as GYPSUM PLASTERS OF HARD-WALL PLASTERS. The base of these products is calcium sulphate or gypsum which has been calcined to partially expel the water. The setting and hardening of these products is dependent upon their combining chemically with the gauging water and crystallizing in the same chemical form as the material possessed before calcination. All hard-wall plasters contain material added for the purpose of controlling the SET. The straight calcined gypsum sets in a very few minutes, which time would be entirely too short to permit the workmen to apply the plaster to the wall and straighten it up before it had set. These plasters are characterized, also, by their inability to carry as much sand as lime mortar. Many of them contain other substances, such as clay or hydrated lime, added to improve their PLASTICITY. Hard-wall plasters manufactured in the eastern part of the United States from rock-gypsum invariably contain 15%, more or less, of clay or hydrate, added for this purpose. Plasters made in Kansas, Oklahoma, Texas and other

Western and Southwestern States are made from earth-gypsum. In the case of these materials, clay and hydrated lime are not added, for the reason that the earth-gypsum contains considerable clay matter, which renders further additions unnecessary.

Use of Hard-Wall Plasters. Hard-wall plasters are found to be very convenient in cases where space and time are the most important elements in the building operation. They set more rapidly than lime plasters, thus permitting the white coating and finishing of the job to be completed earlier. While hard-wall plasters become extremely hard, this property is sometimes considered objectionable, as it may give rise to what is called the SOUNDING-BOARD effect.

Keene's Cement Plasters. As distinguished from the ordinary hard-wall plasters, there exists another class of gypsum-products which, however, are somewhat different in the method of preparation and behavior. In the manufacture of these materials, the gypsum is calcined, immersed in a bath of alum or similar chemical and recalcined. The name KEENE'S CEMENT is usually applied to these materials, which are made by several manufacturers in this country. These are slow-setting and ultimately attain great strength and hardness. Keene's cement is generally used with considerable lime-putty or hydrated lime. The use of equal parts of hydrated lime and Keene's cement in making a plastering material is often recommended and found in specifications. (For Alca lime used as a wall-plaster, see page 1553.)

Advantages of Improved Wall-Plasters. Among the advantages gained by the use of these plasters are uniformity in strength and quality, extra hardness and toughness, freedom from pitting, saving in time required in making and drying, minimum danger from frost while being applied and before set, less weight and moisture in the building, and, in some cases, greater resistance to the action of fire.

Measuring Plasterers' Work. Lathing is always figured by the square yard and is generally included with the plastering, although in small country towns the carpenter often puts on the laths. Plastering on plane surfaces, such as walls and ceilings, is always measured by the square yard, whether it is one-coat, two-coat, or three-coat work, or lime or hard plaster. In regard to deductions for openings, custom varies somewhat in different parts of the country and also with different contractors. Some plasterers allow one-half the area of openings for ordinary doors and windows, while others make no allowance for openings of less than 7 sq yd.

Miscellaneous Details. Returns of chimney-breasts, pilasters and all strips less than 12 in in width should be measured as 12 in wide. Closets, soffits of stairs, etc., are generally figured at a higher rate than plain walls or ceilings, as it is not as easy to get at them. For circular or elliptical work, domes or groined ceilings, an additional price is made. If the plastering cannot be done from trestles an additional charge must be made for staging. Whenever plastering is done by measurement the contract should definitely state whether or not openings are to be deducted, and a special price should be made for the stucco-work, based on the full-size details.

Cornices and Moldings. Stucco cornices and molded work are generally measured by the superficial foot, measuring on the profile of the molding. When less than 12 in in girth they are usually rated as 1 ft. For each internal angle 1 lin ft should be added, and for external angles, 2 lin ft. For cornices on circular or elliptical work an additional price should be charged. Enriched moldings are generally figured by the linear foot, the price depending upon the design and size of the mold.

Quantities of Materials for Lathing and Plastering

Miscellaneous Data. To cover 100 sq yd requires from 1 400 to 1 500 laths, or say 1 450 for an average job, and 10 lb of threepenny fine nails.

Three-coat plastering on wooden laths, plaster-of-Paris finish, will require from 10 to 12 bu of lime, 1½ cu yd of sand, 2 bu of hair and 100 lb of plaster of Paris per 100 sq yd.

If the finish-coat is omitted, deduct 2 bu of lime and all of the plaster of Paris.

If sand-finished, omit the plaster of Paris and add ½ cu yd of sand.

To cover 100 sq yd with two coats on brick or stone walls, the brown coat and finishing coats, will require from 8 to 10 bu of lime, 1½ cu yd of sand, and 100 lb of plaster of Paris, to 100 sq yd.

Using Best's Keene's cement for brown mortar and Keene's finish on expanded-metal lath will require, for brown mortar, 550 lb of cement, 5½ bu of lime, 2 cu yd of sand and 2 bu of hair; for the finish, 300 lb of cement and 1 bu of lime per 100 yd.

Hard plasters on expanded-metal lath, plaster-of-Paris finish, require, for brown mortar, 2 000 lb of plaster and 2 cu yd of sand; for the finish, 1 bu of lime and 100 lb of plaster of Paris per 100 yd.

Cost of Lathing and Plastering. The average price for putting on wooden laths, labor only, is 4¾ cts per yard. For expanded or sheet-metal laths on wooden studding, 5¾ cts; on steel studding, wired, from 10 to 12 cts.

The cost of putting three coats on laths, plaster-of-Paris finish, labor only, runs about 22 cts per yard.

With sand finish the cost is about 23 cts.

These figures are based on plasterers' wages at 75 cts per hour, and 50 cts per hour for hod-carriers and mortar mixers.

The following schedule * gives the average cost of different kinds of plastering, based on lime at 40 cts per bushel, sand at 75 cts per load of 1¼ cu yd, hair at 40 cts per bushel, plaster of Paris at 50 cts per 100 lb.

Scratch and brown coat (lime) on wooden laths	25 cts per sq yd.
Three coats (lime) on wooden laths, plaster-of-Paris finish	30 cts per sq yd.
Three coats (lime) on wooden laths, sand finish	30 cts per sq yd.
Brown coat and finish on brick walls	23 cts per sq yd.
For hard-wall plaster instead of lime, add	3 cts per sq yd.
Three coats (lime), plaster-of-Paris finish, metal lath on wooden studding	65 cts per sq yd.
Three coats (lime) plaster-of-Paris finish, metal lath on steel studding	68 cts per sq yd.
For Keene's cement finish, add	10 cts per sq yd.
For blocking in imitation of tile, add	50 cts per sq yd.
Two coats hard-wall plaster, plaster-of-Paris finish, metal lath, wooden studding	70 cts per sq yd.
Two coats hard-wall plaster, plaster-of-Paris finish, metal lath on steel studs	73 cts per sq yd.
For Keene's cement finish, add	10 cts per sq yd.
Portland cement, brown coat, finished with Keene's cement blocked in imitation of tile, 3 by 6 in	$2.80 per sq yd.
For running base, 9 in high, in Best's Keene's cement	10 cts per ft.

For running plain moldings in plaster of Paris, from 3 to 5 cts per inch of girth.

For finishing shafts of columns, from 16 to 24 in in diam., from 12 to 14 ft high, $3 per column (labor only).

* These are pre-war prices and the unit values per sq yd must be largely increased on account of the increase in wages and materials.

These prices, of course, vary somewhat in different sections of the country. In some localities prices for materials or labor are less, in others higher.

Staff is a composition of plaster of Paris and hemp-fiber, cast in molds and nailed or wired in place. All of the buildings of the Columbian Exposition at Chicago (1893) were covered with this material and all of the temporary buildings of the St. Louis Exposition (1904). It is not sufficiently durable for permanent work unless it is frequently painted. The cost of staff, as used on the buildings at Chicago in 1893, varied from $2 to $2.25 per sq yd.

DATA ON LUMBER AND CARPENTERS' WORK*

Relative Hardness of Woods. Taking shell-bark hickory as the highest standard of our forest-trees, and calling that 100, other trees will compare with it for hardness as follows:

Shell-bark hickory	100	Yellow oak	60
Pignut hickory	96	Hard maple	56
White oak	84	White elm	58
White ash	77	Red cedar	56
Dogwood	75	Wild cherry	55
Scrub-oak	73	Yellow pine	54
White hazel	72	Chestnut	52
Apple-tree	70	Yellow poplar	51
Red oak	69	Butternut	43
White beech	65	White birch	43
Black walnut	65	White pine	30
Black birch	62		

Weight of Rough Lumber per 1 000 Feet

BOARD-MEASURE, APPROXIMATE

For weight of various woods see tables on pages 1501 to 1508

Kind of wood	Green from saw, lb	Shipping-dry, lb	Well-seasoned, lb	Kiln-dried, lb
Ash				
Chestnut	4 600	....	3 500	3 200
Hemlock	4 200	3 000	....	......
Maple, hard	5 400	4 150	3 900	3 400
Maple, soft	5 000	3 650	3 300	3 000
Oak, red	5 500	4 250	4 000	3 400
Oak, white	5 700	4 500	4 100	3 600
Pine, long-leaf	4 500	3 500	....	....
Pine, white	3 500	2 500	2 400	2 200
Poplar	4 000	3 000	2 900	2 400
Spruce	3 150	2 700	2 300	2 200
Sycamore	4 750	3 200	3 000	......
Walnut, black	4 900	4 000	3 800	....

* A comprehensive booklet giving the rules for the grading and classification of yellow pine lumber and dressed stock may be obtained from The Southern Pine Association, New Orleans, La.

Framing-Lumber may commonly be purchased in any of the following nominal sizes, except that common pine, spruce, and hemlock cannot usually be obtained in larger sizes than 12 by 12 in.

Nominal Sizes of Framing-Lumber

in	in	in	in
2 × 4	3 × 6	4 × 12	8 × 12
2 × 6	3 × 8	4 × 14	8 × 14
2 × 8	3 × 10	6 × 6	10 × 10
2 × 10	3 × 12	6 × 8	10 × 12
2 × 12	3 × 14	6 × 10	10 × 14
2 × 14	3 × 16	6 × 12	10 × 16
2 × 16	4 × 4	6 × 14	12 × 12
2½ × 12	4 × 6	6 × 16	12 × 14
2½ × 14	4 × 8	8 × 8	12 × 16
2½ × 16	4 × 10	8 × 10	14 × 14
....	....	....	14 × 16

In some of the New England mills, the following sizes, also, are sawed: 2 by 3, by 5, 2 by 7, 2 by 9, 3 by 4 and 3 by 5 in. These sizes are not commonly carried n stock, and in most localities would have to be obtained by ripping larger izes. Most of the long-leaf yellow pine and Douglas fir is SHIPPED SURFACED NE SIDE AND EDGE, the actual dimensions being from ¼ in to ⅜ in, and someimes ½ in, scant of the nominal dimensions. When framing-lumber is required o be full to dimensions it should be ordered IN THE ROUGH, and a special contract nade on that understanding.

Lengths of Framing-Timbers. All timber is cut and sold in even lengths, as o, 12, 14 and 16 ft. Odd and fractional lengths are counted as the next higher ven length; consequently it is, in certain cases, possible and economical to plan uildings so that timbers of even lengths may be used without waste.

Measurement of Rough Lumber. All rough lumber is sold by the foot, OARD-MEASURE, one foot being the equivalent of a board 1 ft wide, 1 ft long, and in thick. To compute the board-measure in any board, plank, or timber, divide the nominal sectional area, in inches, by 12, and multiply by the length in eet. Thus the number of FEET in a 2 by 4-in scantling, 8 ft long = $(2 \times 4/12) \times 8 = 5\frac{1}{3}$ ft, board-measure. A 10-in board, 12 ft long, contains $(1 \times 10/12) \times 12 = 10$ ft, board-measure. Extensive tables are published showing the feet, in oard-measure, for almost any commercial size of timber. The following table, owever, although compact, will enable one to readily estimate the number of EET in any of the standard sizes of boards, planks, or timbers. To use the table, nd the product of the lateral dimensions of the cross-section; then in the olumn having a heading equal to this product, and in the horizontal line opposite the given length will be found the number of feet in board-measure. Thus, or a 3 by 4, 2 by 6, or 1 by 12-in timber look in the column headed 12; for a 2 y 12, 4 by 6, or 3 by 8-in piece, look in the column headed 24. For lengths not iven in the table, take either twice the length and divide by 2, or one-half the ength and multiply by 2. Where timbers of the same size abut end to end, it conomizes labor in reducing to board-measure to take the full length; for this eason the lengths in the table are carried beyond those for single sticks.

Table of Board-Measure

For explanation, see page 1559

Length in feet	Sectional area in square inches: 4		6	8		10		12	14		16		18	20	
	ft	in	ft*	ft	in	ft	in	ft *	ft	in	ft	in	ft *	ft	in
6	2	0	3	4	0	5	0	6	7	0	8	0	9	10	0
8	2	8	4	5	4	6	8	8	9	4	10	8	12	13	4
10	3	4	5	6	8	8	4	10	11	8	13	4	15	16	8
12	4	0	6	8	0	10	0	12	14	0	16	0	18	20	0
14	4	8	7	9	4	11	8	14	16	4	18	8	21	23	4
16	5	4	8	10	8	13	4	16	18	8	21	4	24	26	8
18	6	0	9	12	0	15	0	18	21	0	24	0	27	30	0
20	6	8	10	13	4	16	8	20	23	4	26	8	30	33	4
22	7	4	11	14	8	18	4	22	25	8	29	4	33	36	8
24	8	0	12	16	0	20	0	24	28	0	32	0	36	40	0
26	8	8	13	17	4	21	8	26	30	4	34	8	39	43	4
28	9	4	14	18	8	23	4	28	32	8	37	4	42	46	8
30	10	0	15	20	0	25	0	30	35	0	40	0	45	50	0
32	10	8	16	21	4	26	8	32	37	4	42	8	48	53	4
34	11	4	17	22	8	28	4	34	39	8	45	4	51	56	8
36	12	0	18	24	0	30	0	36	42	0	48	0	54	60	0
38	12	8	19	25	4	31	8	38	44	4	50	8	57	63	4
40	13	4	20	26	8	33	4	40	46	8	53	4	60	66	8
42	14	0	21	28	0	35	0	42	49	0	56	0	63	70	0

Length in feet	Sectional area in square inches: 24	28		30	32		35		36	40		42	48
	ft *	ft	in	ft *	ft	in	ft	in	ft *	ft	in	ft *	ft *
6	12	14	0	15	16	0	17	6	18	20	0	21	24
8	16	18	8	20	21	4	23	4	24	26	8	28	32
10	20	23	4	25	26	8	29	2	30	33	4	35	40
12	24	28	0	30	32	0	35	0	36	40	0	42	48
14	28	32	8	35	37	4	40	10	42	46	8	49	56
16	32	37	4	40	42	8	46	8	48	53	4	56	64
18	36	42	0	45	48	0	52	6	54	60	0	63	72
20	40	46	8	50	53	4	58	4	60	66	8	70	80
22	44	51	4	55	58	8	64	2	66	73	4	77	88
24	48	56	0	60	64	0	70	0	72	80	0	84	96
26	52	60	8	65	69	4	75	10	78	86	8	91	104
28	56	65	4	70	74	8	81	8	84	93	4	98	112
30	60	70	0	75	80	0	87	6	90	100	0	105	120
32	64	74	8	80	85	4	93	4	96	106	8	112	128
34	68	79	4	85	90	8	99	2	102	113	4	119	136
36	72	84	0	90	96	0	105	0	108	120	0	126	144
38	76	88	8	95	101	4	110	10	114	126	8	133	152
40	80	93	4	100	106	8	116	8	120	133	4	140	160
42	84	98	0	105	112	0	122	6	126	140	0	147	168

* The measurements in these columns come out in even feet.

Table of Board-Measure (Continued)

For explanation, see page 1559

Length in feet	Sectional area in square inches: 56 ft	56 in	60 ft *	64 ft	64 in	72 ft *	80 ft	80 in	84 ft *	96 ft *	100 ft	100 in	112 ft	112 in
4	18	8	20	21	4	24	26	8	28	32	33	4	37	4
6	28	0	30	32	0	36	40	0	42	48	50	0	56	0
8	37	4	40	42	8	48	53	4	56	64	66	8	74	8
10	46	8	50	53	4	60	66	8	70	80	83	4	93	4
12	56	0	60	64	0	72	80	0	84	96	100	0	112	0
14	65	4	70	74	8	84	93	4	98	112	116	8	130	8
16	74	8	80	85	4	96	106	8	112	128	133	4	149	4
18	84	0	90	96	0	108	120	0	126	144	150	0	168	0
20	93	4	100	106	8	120	133	4	140	160	166	8	186	8
22	102	8	110	117	4	132	146	8	154	176	183	4	205	4
24	112	0	120	128	0	144	160	0	168	192	200	0	224	0
26	121	4	130	138	8	156	173	4	182	208	216	8	242	8
28	130	8	140	149	4	168	186	8	196	224	233	4	261	4
30	140	0	150	160	0	180	200	0	210	240	250	0	280	0
32	149	4	160	170	8	192	213	4	224	256	266	8	298	8
34	158	8	170	181	4	204	226	8	238	272	283	4	317	4
36	168	0	180	192	0	216	240	0	252	288	300	0	336	0
38	177	4	190	202	8	228	253	4	266	304	316	8	354	8
40	186	8	200	213	4	240	266	8	280	320	333	4	373	4
42	196	0	210	224	0	252	280	0	294	336	350	0	392	0
44	205	4	220	234	8	264	293	4	308	352	366	8	410	8
46	214	8	230	245	4	276	306	8	322	368	383	4	429	4
48	224	0	240	256	0	288	320	0	336	384	400	0	448	0
50	233	4	250	266	8	300	333	4	350	400	416	8	466	8
52	242	8	260	277	4	312	346	8	364	416	433	4	485	4
54	252	0	270	288	0	324	360	0	378	432	450	0	504	0
56	261	4	280	298	8	336	373	4	392	448	466	8	522	8
58	270	8	290	309	4	348	386	8	406	464	483	4	541	4
60	280	0	300	320	0	360	400	0	420	480	500	0	560	0
62	289	4	310	330	8	372	413	4	434	496	516	8	578	8
64	298	8	320	341	4	384	426	8	448	512	533	4	597	4
66	308	0	330	352	0	396	440	0	462	528	550	0	616	0
68	317	4	340	362	8	408	453	4	476	544	566	8	634	8
70	326	8	350	373	4	420	466	8	490	560	583	4	653	4
72	336	0	360	384	0	432	480	0	504	576	600	0	672	0
74	345	4	370	394	8	444	493	4	518	592	616	8	690	8
76	354	8	380	405	4	456	506	8	532	608	633	4	709	4
78	364	0	390	416	0	468	520	0	546	624	650	0	728	0
80	373	4	400	426	8	480	533	4	560	640	666	8	746	8
82	382	8	410	437	4	492	546	8	574	656	683	4	765	4
84	392	0	420	448	0	504	560	0	588	672	700	0	784	0

* The measurements in these columns come out in even feet.

Table of Board-Measure (Continued)

For explanation, see page 1559

Length in feet	120 10×12 ft *	140 10×14 ft	in	144 12×12 ft *	160 10×16 ft	in	168 12×14 ft *	192 12×16 ft *	196 14×14 ft	in	224 14×16 ft	in
4	40	46	8	48	53	4	56	64	65	4	74	8
6	60	70	0	72	80	0	84	96	98	0	112	0
8	80	93	4	96	106	8	112	128	130	8	149	4
10	100	116	8	120	133	4	140	160	163	4	186	8
12	120	140	0	144	160	0	168	192	196	0	224	0
14	140	163	4	168	186	8	196	224	228	8	261	4
16	160	186	8	192	213	4	224	256	261	4	298	8
18	180	210	0	216	240	0	252	288	294	0	336	0
20	200	233	4	240	266	8	280	320	326	8	373	4
22	220	256	8	264	293	4	308	352	359	4	410	8
24	240	280	0	288	320	0	336	384	392	0	448	0
26	260	303	4	312	346	8	364	416	424	8	485	4
28	280	326	8	336	373	4	392	448	457	4	522	8
30	300	350	0	360	400	0	420	480	490	0	560	0
32	320	373	4	384	426	8	448	512	522	8	597	4
34	340	396	8	408	453	4	476	544	555	4	634	8
36	360	420	0	432	480	0	504	576	588	0	672	0
38	380	443	4	456	506	8	532	608	620	8	709	4
40	400	466	8	480	533	4	560	640	653	4	746	8
42	420	490	0	504	560	0	588	672	686	0	784	0
44	440	513	4	528	586	8	616	704	718	8	821	4
46	460	536	8	552	613	4	644	736	751	4	858	8
48	480	560	0	576	640	0	672	768	784	0	896	0
50	500	583	4	600	666	8	700	800	816	8	933	4
52	520	606	8	624	693	4	728	832	849	4	970	8
54	540	630	0	648	720	0	756	864	882	0	1 008	0
56	560	653	4	672	746	8	784	896	914	8	1 045	4
58	580	676	8	696	773	4	812	928	947	4	1 082	8
60	600	700	0	720	800	0	840	960	980	0	1 120	0
62	620	723	4	744	826	8	868	992	1 012	8	1 157	4
64	640	746	8	768	853	4	896	1 024	1 045	4	1 194	8
66	660	770	0	792	880	0	924	1 056	1 078	0	1 232	0
68	680	793	4	816	906	8	952	1 088	1 110	8	1 269	4
70	700	816	8	840	933	4	980	1 120	1 143	4	1 306	8
72	720	840	0	864	960	0	1 008	1 152	1 176	0	1 344	0
74	740	863	4	888	986	8	1 036	1 184	1 208	8	1 381	4
76	760	886	8	912	1 013	4	1 064	1 216	1 241	4	1 418	8
78	780	910	0	936	1 040	0	1 092	1 248	1 274	0	1 456	0
80	800	933	4	960	1 066	8	1 120	1 280	1 306	8	1 493	4
82	820	956	8	984	1 093	4	1 148	1 312	1 339	4	1 530	8
84	840	980	0	1 008	1 120	0	1 176	1 344	1 372	0	1 568	0

* The measurements in these columns come out in even feet.

Measurement of Finishing-Lumber, Flooring, Ceiling, Etc. Most, if not l, lumber for finishing is sawed for use in thicknesses of 1 in, 1¼ in, 1½ in, and in, and some woods, such as white pine and poplar, are sawed into thicknesses 2½ in and 3 in.

When surfaced both sides, the thickness is reduced to 13/16, 1 1/16, 1 5/16, 1¾, 2¼, d 2 11/16 in.

All dressed stock is measured and sold STRIP-COUNT, that is, full size of rough aterial necessarily used in its manufacture. Thus 1 1/16-in boards are measured though 1¼ in thick. The number of feet, board-measure, for 1¼-in stock 1/16 finished) is 1¼ times that in a 1-in board, and in the same way for 1½-in d 2½-in stock. 1¾-in planks are always measured 2 in thick, and 2¼-in ock, 2½ in thick. Boards less than 1 in thick are measured the same as 1-in ards, but for ⅜-in and ⅝-in stock a reduced price is generally made.

Matched Ordinary Flooring.* The standard sizes for flooring (other than rdwood, parqueting or parquet-flooring) are 1 by 3, 1 by 4 and 1 by 6; or 1¼ by 1¼ by 4 and 1¼ by 6. The thickness of 1-in flooring should be 13/16 in, and 1¼-in flooring, 1 3/32 in. 3-in flooring should show 2¼ in on the face, after it is d; 4-in, 3¼ in; and 6-in, 5¼ in.

Matched Maple Flooring is usually made in 2-in, 2¼-in and 3¼-in face, and thicknesses of 13/16, 1 1/16 and 1 5/16 in.

Ceiling, matched and beaded boards, is regularly stuck in the same widths as oring. The standard (nominal) thicknesses of yellow-pine ceiling are ⅜, ½, and ¾ in, the actual thickness of each being 1/16 in less. The ⅜-in ceiling is essed one side only, the other thicknesses both sides.

Yellow Pine Drop-Siding. Dressed and matched yellow pine drop-siding is by 3½ and ¾ by 5½ in, showing 3¼ and 5¼-in face; and worked shiplap is by 3½ and ¾ by 5½ in, showing 3 and 5-in face.

Beveled Siding is resawed on a bevel from stock 13/16 by 3½ and 13/16 by 5½ in, er surfacing.

New England Clapboards are 4 ft long, 6 in wide, ½ in thick at the butt, and out ⅛ in thick at the other edge. They are put up in bunches and sold by the ousand.

Rules for Estimating Quantities of Sheathing, Flooring, Etc. For com- n sheathing laid horizontally on a wall or roof without openings, add one- th to the actual superficial area to allow for waste. On the walls of dwellings, ure the walls as though without openings and allow nothing for waste. If athing is laid diagonally, add one-sixth to the actual superficial area.

For tight sheathing laid horizontally, add one-fifth for 6-in boards, one-seventh 8-in boards, and one-ninth for 10-in boards. If laid diagonally add one- rth for 6-in boards, one-sixth for 8-in boards, and one-eighth for 10-in boards.

For 3-in matched flooring add one-half to the actual superficial area to be vered.

For 4-in flooring add one-third and for 6-in flooring add one-fifth. Ceiling is asured the same as flooring.

For drop-siding, add one-fifth to the superficial area.

For lap-siding, laid 4 in to the weather, add one-half to the actual superficial a; if 4½ in to the weather, add one-third.

* Everywhere except in New England FLOORING is always understood to be tongued and oved.

Cost of Labor for Carpenters' Work. There are so many items and co ditions which enter into the cost of carpenters' work, and the cost varies widely with the locality, that it is quite impossible to give figures which are general practical value, although several books * have been published on es mating labor and materials for buildings.

The following figures of the cost,† for labor and nails, of framing and putti on sheathing and siding and laying flooring were computed on the basis of ca penters' wages at $3 a day of eight hours (37½ cts per hour). The cost framing is almost always figured at a certain price per thousand feet of lumb board-measure. The cost of laying flooring, sheathing, etc., is almost alw figured by the square of 100 sq ft (10 by 10 ft).

Character of work	Cost
For setting up studding and framing walls of wooden dwellings	$10.00 per 1000
For framing and setting floor-joists, 2 by 8 to 2 by 12	$9.00 to $10.00 per 1000
Framing and setting heavy joists and girders, 6 by 12 to 10 by 14	$ 8.50 per 1000
Framing gable roofs and setting in place	10.00 per 1000
Framing hip-roofs and setting in place	$11.00 to $12.00 per 10
For putting in bridging, after it is cut, per 100 lin ft in the row	$1.25
For covering the sides or roofs of wooden buildings with dressed sheathing, laid horizontally	60 cts per square
The same, if laid diagonally	75 cts per square
The cost of labor and nails for laying 6-in flooring, blind-nailed to every joist, without dressing after laying, is about	$2.00 per square
For 4-in flooring, not dressed, allow	2.25 per square
For 3-in flooring, not dressed, allow	2.50 per square
For 3-in hard-pine flooring, hand-smoothed or traversed.	3.75 per square
For 3-in red-oak flooring, hand-smoothed or traversed	6.00 per square
For 3-in white-oak flooring, hand-smoothed or traversed	8.00 per square
For 3-in maple flooring, hand-smoothed or traversed	$10.00 to $12.00 per sq

BUILDING PAPERS, BUILDING FELTS AND QUILTS

Sheathing-Papers,‡ Felts, Quilts, Etc. It is well known that frame bui ings when merely sheathed and clapboarded or shingled on the outside and simpl lathed and plastered on the inside, are almost sure to be hot in summer and co in winter; and as the wood almost always shrinks, cracks result through whi the wind finds its way. For these reasons some extra provision should be mad for keeping out the wind and the heat and cold; and it is generally admitted th

* Readers are referred to The Building Estimator's Reference Book, by F. R. Walk The New Building Estimator, by William Arthur, Handbook of Cost Data, by H Gillette and the Estimators' Price Book, by I. P. Hicks. To all of these, architects a builders are referred for detailed information and valuable data on costs of labor a material.

† The wages of carpenters varied (1916) in the United States from 35 to 70 cts per ho or from $2.80 to $5.60 per day of 8 hours. For rates per day higher than those giv the figures showing the costs in the schedule must be raised proportionately.

‡ The terms BUILDING PAPER and SHEATHING-PAPER are by the public indiscriminat applied to all kinds of paper used in connection with building-construction. In t trade, however, the term BUILDING PAPER is confined to the rosin-sized and chea grades of paper, while the heavier and better grades are classed as SHEATHING-PAPERS.

ere is no material that will do this so well and at so small an expense as good athing-papers or sheathing-felts. The papers made for this purpose are comnly known as SHEATHING-PAPERS or BUILDING PAPERS. There is a great riety of sheathing-papers manufactured, many of them of great excellence, d even the best are comparatively inexpensive, costing only about $1.00 per o sq ft; so that only the better qualities of any kind of felt or paper should be ecified. Where the cost of the sheathing-paper on an ordinary house is only ew dollars, it is poor economy to use a cheap paper, as the labor of applying it an important item and the poorer the paper the more difficult the work of putg it on. The qualities which good sheathing-paper should possess are permence, impenetrability to air and water and sufficient strength to permit of plying without tearing. Protection or proof against vermin and insects is other important requirement. It should not be brittle nor have a lasting ong odor and, for the convenience of the builder, should be clean for handling. ere are so many papers possessing all or most of these qualities that it is deemed xpedient to mention particular brands. The architect should decide for himf, from the samples with which he has probably been furnished, what papers best adapted to the particular conditions; and he should then specify those ands, giving, also, the manufacturers' names, instead of leaving the choice to builder, who will be quite sure to be guided by price rather than by quality. any object to tarred or saturated sheathing-papers and felts because of their idency to become brittle and because they emit a strong odor and are someat disagreeable to handle. On the other hand, the advocates of tarred felts phasize their cheapness, warmth and even their odor, which makes them rmin-proof. The odor gradually disappears after the clapboards, siding or ngles are put on and the inside walls finished. Sheathing-paper is usually plied just previous to putting on the clapboards, siding, or shingles. It is nerally placed horizontally and should lap about 2 in over each sheet and over paper previously placed around the window and door-frames. If sheathinglt or similar material is to be placed under the clapboards or siding, laths ould be nailed vertically over it, opposite each stud, and the siding or claprds nailed to the laths; otherwise it will be difficult to put them on evenly, ing to the thickness and elastic quality of the QUILT. Shingles, however, may applied directly over it. Sheathing-quilt possesses marked fire-resisting perties. The sheathing-paper and the labor of putting it on should be inded in the carpenter's specifications.

Rosin-Sized Building-Papers. These are the common grades of building per; they are not water-proof, and should not be used on roofs or on walls in np climates. In dry places they protect from dust, draughts, and to some ent from heat and cold. They are generally either a dull red or gray in color, re a hard, smooth surface, and are clean to handle. They are always put up rolls 36 in wide and usually contain 500 sq ft. The weight varies from 18 to lb to the roll of 500 sq ft.

Insulating and Deadening-Quilts. Among the insulating and deadeninglts much in use are those mentioned below. There are also other good maals in this line which are manufactured and used for insulating and deadening poses.

Sheathing-Quilt.* This consists of a felted matting of eel-grass held in place ween two layers of strong Manila paper by quilting. "The long, flat fibers eel-grass cross each other at every angle and form within each layer of quilt umerable minute dead-air spaces, that make a soft, elastic cushion. This

* Made by Samuel Cabot (Inc.), Boston, Mass.

gives the most perfect conditions for non-conduction." Eel-grass is cho for the filling because of its long, flat fibers, which especially adapt it f felting; because of its great durability,* and its resistance to fire; and beca owing to the large percentage of iodine which it contains, it is repellent to n and vermin. This quilt is made in single and double-ply thickness, and is p up in bales of 500 sq ft. It is also now made with a covering of asbestos, whi tenders it thoroughly fire-proof. The material is also very efficient for he insulation. When used for this purpose there is no objection to nails passi through it.

Keystone Hair Insulator. Another material used for similar purposes is t Keystone Hair Insulator.† This consists of thoroughly cleansed cattles' ha between two layers of strong, non-porous building paper, securely stitched t gether. The hair is chemically treated, so that it is coated with lime, whi makes the finished material vermin-proof and odorless.

Mineral-Wool Deadeners, which are fire-proof sound-deadening quilts rock-fiber wool stitched between two sheets of building paper or of asbest paper according to the grade desired, are made by the Union Fibre Company Winona, Minn., and other firms. This company makes, also, what is call Lith and Feltlino, which are sound-deadening materials in board form. Th manufacture, also, Linofelt, a building-quilt of flax-fibers (unbleached lin threads), stitched between water-proof paper or asbestos paper according to nee It is ¼ in thick. Linofelt for sheathing in place of ordinary building paper ad from 1 to 1½% to the cost of a house.

Felt-Papers. There are a great many felt-papers for lining floors and a fe are made fire-proof by means of chemicals. As a rule these felts are cheaper th Cabot's QUILT, although the saving in an ordinary residence would be but littl and even among the felts themselves there is quite a difference in cost. I choosing a felt-paper for lining, the architect should select one that is soft a elastic enough to form a cushion, and the thicker the felt, provided it has th above qualities, the greater will be its non-conduction. Some felts are ma water-proof by an asphalt center, which is an advantage in case of fire or leak but some authorities think that it is doubtful if such felts obstruct the passage sound as well as felts without the asphalt center. The experience of so acoustical experts seems to show that one of the best methods of deadening is t a combination of heavy hair-felt or felt-paper with sheets of galvanized iro Two layers of felt, each from ½ to 1 in thick, are placed on either side of a sing layer of galvanized iron, the latter resting freely between the felt layers. Th form of construction is to be preferred where the deadening-material is not a tached to the enclosing woodwork. An additional layer of iron and of fe increases the effectiveness of the combination.

Saturated Felts.‡ Common roofing-felts are made by saturating comm dry felt with coal-tar pitch. Roofing-felts are commonly made in weights of 1 15, and 20 lb to the 100 sq ft. Nothing lighter than 12 lb should be used f roofing. They are usually sold by weight. Asphalt-felts are commonly ma in the same weights.

Dry Saturated Tarred Felts are specially run through a tier of calend to give a hard, uniform surface and contain a minimum amount of coal-ta

* A sample of eel-grass 250 years old and in a perfect state of preservation, may be se at Mr. Cabot's office.

† Made by Johns-Manville, Inc., New York.

‡ The Barrett Company and other manufacturers make numerous brands of the sheathing and roofing-papers.

ey are especially adapted for slaters' use, as they will carry a chalk line and e easy to handle. The rolls are 36 in wide, contain 500 sq ft and weigh about lb.

Asbestos Building Felts are usually made about 6, 10, 14 and 16 lb to the o sq ft, although different manufacturers make different weights. They come rolls 36 in wide and are sold by weight.

Sound-Deadening Felts. These deadening-felts are made by various manuturers. In one of these felts * the material itself is rather hard and thin, but s pressed in such a way as to form small indentations or air-cells. This makes lastic and breaks up the sound-waves.

Asbestos Sheathing. Sheathing-papers or building felts, made of asbestos, used to a considerable extent for floor-linings and for covering the outside lls of wooden buildings, principally on account of their fire-proof and verminof qualities. These papers are well known in the trade and can be procured hout difficulty. They are supplied by the manufacturers in 50 or 100-lb s, 36 in wide, on a basis of the following scale of weights:

4 lb to the 100 sq ft	18 lb to the 100 sq ft
6 lb to the 100 sq ft	20 lb to the 100 sq ft
8 lb to the 100 sq ft	24 lb to the 100 sq ft
10 lb to the 100 sq ft	32 lb to the 100 sq ft
12 lb to the 100 sq ft	1/16 in thick
14 lb to the 100 sq ft	3/32 in thick
16 lb to the 100 sq ft	1/8 in thick

he sheathing in the 1/16, 3/32 and 1/8-in thicknesses is used only for special pures where an unusually thick lining is desired for possible fire-protection und exposed flues, for chimney-breasts, etc. When the weight of paper exls 32 lb to the square foot it is known as ROLL-BOARD and is no longer classed weight per 100 sq ft, but by thickness. For floor-linings, 16-lb paper is genly employed, this weight being sufficiently thick and strong to resist ordiy damage in application and in handling. Asbestos felts and building papers ear to have approximately the same effect in retarding the passage of soundes as other felt-papers of a relatively similar thickness and quality, while r fire-proof and vermin-proof qualities are a distinct advantage. The cost of stos paper and building-felt, while somewhat greater than that of the ordi- papers used for similar purposes, is not excessive. The market price varies depends upon the fluctuations of the market. For example, the cost of sq ft of 16-lb asbestos paper varied from 32 to 40 cts, according to the ket, before the war.†

ater-Proof Papers. Neponset Black Sheathing is water-proof and roof, odorless and clean to handle, and is an excellent paper under g, shingles, slate, or tin. The rolls are 36 in wide, containing 250 and sq ft.

eponset Red Rope Sheathing and Roofing. This is made of rope-, has great strength and flexibility, and is absolutely water-proof and air-. It is one of the best sheathing-papers and makes a good cheap roofing heds, poultry-houses, etc. The rolls are 36 in wide, containing 100, 250 and sq ft.

Neponset Florian Sound-Deadening Felt, supplied by Bird & Son, East Walpole,

† These prices are now much higher.

Parchment Water-Proof Sheathing. There are various parchment-sheat ings on the market which are semitransparent, have smooth surfaces, and odorless, water-proof, air-proof and vermin-proof. They are adapted for gen sheathing purposes. In general 1-ply weighs 25 lb to 900 sq ft; 2-ply, 25 lb 500 sq ft; 3-ply, 25 lb to 275 sq ft. They are 36 in wide.

Cost of Building and Sheathing-Papers in Place.* The following, though necessarily restricted to a few lines, will give a general idea of the cost different kinds and grades of sheathing-papers, the prices given being fair av ages for the materials APPLIED to an outside wall or roof:

	Price per 100 square feet
Common tarred felts (15 lb per square)	30 cts
Red rosin-sized sheathing, best grades	25 cts
Monahan's parchment-sheathing, single-ply	26 cts
Monahan's parchment-sheathing, double-ply	40 cts
Monahan's ship-rigging tar-sheathing, 2-ply	75 cts
"Neponset" black (water-proof) paper	45 cts
"Neponset" red-rope roofing	$1.20
Sheathing-papers with asphalt center	40 to 50 cts
Asbestos building or sheathing-felt, 10 lb per square	22½ cts
Asbestos building or sheathing-felt, 14 lb per square	31½ cts
Cabot's sheathing-quilt, single-ply	$1.05
Cabot's sheathing-quilt, double-ply	$1.25
Barrett's specification-felt	35 cts
Barrett's DEFENDER, felt-sheathing	80 cts
Sackett's water-proof sheathing	30 cts
Empire parchment-sheathing, 1-ply	25 cts
Empire parchment-sheathing, 2-ply	36 cts
Empire parchment-sheathing, 3-ply	50 cts
Barrett's red rope	$1.00
Barrett's black, water-proof sheathing	40 cts

PAINT AND VARNISH †

Pigments and Vehicles. The solid ingredient of a paint is called the PIGMENT and is a fine powder, nearly all of which will pass through a brass-wire sieve 100 meshes to the linear inch; in fact, most pigments are much finer than this and those formed as precipitates by chemical processes are so fine that there no way to measure them. The liquid part is called the VEHICLE. This is us ally linseed-oil, sometimes with the addition of a little turpentine or other volatil solvent. In the enamel paints it is varnish and in kalsomine and other cold water paints it is a solution of glue, casein, albumen, or some similar cementi material. The cementing material is sometimes called the BINDER.

Ingredients of Oil-Paint. White lead and white zinc are the common whi pigments. There are white pigments of variable composition called leaded zi and zinc lead, furnace-products, composed of zinc oxide and lead sulph There is also a basic lead sulphate, commercially called sublimed white lea which is a similar furnace-product consisting chiefly of sulphate of lea These composite white pigments are largely used in mixed paints. LITHOPO is a mixture of sulphide of zinc and sulphate of barium. It is very white, a

* All prices quoted are pre-war prices and the data are retained for purposes of co parison and relative values.

† The editor is indebted to Professor Alvah H. Sabin for valuable assistance in t ... of the data relating to this subject.

ıd opaque and largely used as the basis of flat wall-finishes for interior work, ıt is not durable for exterior work. It is discolored (grey) by strong light, but ıis is not a very serious practical objection. White lead is used everywhere, ıt tends to yellow somewhat in the dark. White zinc is chiefly used on interior ork, being the whitest paint known. Both are often mixed and both are used mixed paints. Yellow paint is commonly chromate of lead, or chrome yellow; 'een is chrome green, which is a mixture of chrome yellow and Prussian blue; ue is ultramarine, or sometimes Prussian blue. The brilliant reds are coal-tar lors as a rule; the dull reds and browns are oxides of iron. Ochres are dull ellow. Carbon forms the base of all black paints, either as lampblack, drop-ack (boneblack), or graphite. Linseed-oil is either raw or boiled. Raw oil is ıe oil in its natural state as it is extracted from the seed; it should be settled ıd filtered perfectly clear; it is yellow or greenish yellow in color. Boiled oil is w oil which has been heated to 400° or 500° F. with compounds (usually oxides) lead and manganese; it is darker in color than raw oil, and dries quicker. aw oil exposed in a thin film to the air is converted in about five days into a ugh leathery substance; boiled oil undergoes this change in from 10 to 24 hours.

Driers. These are compounds of lead and manganese, dissolved in oil, and is solution thinned with turpentine or benzine. They act as carriers of oxygen tween the air and the oil, and their addition to a paint makes it dry more pidly. Some driers are also called JAPANS. Not more than 10% by volume of y of these liquid driers should be added to oil. Excess of drier causes the int to lack durability. Cheap driers often contain rosin. It is well to specify at driers and japans should be free from ROSIN (not resin, as varnish-resins are esent in some of the best driers).

Priming Coat. This is the first coat applied to the clean surface. A priming at for wood is chiefly oil, and is usually equivalent to a gallon of ordinary paint inned with a gallon of raw linseed-oil. Paint, however, is not thinned to make priming coat for structural metal. In all wood-work, nail-holes and other de-ts are filled with putty after the priming coat has been applied; but if the od is resinous, knots and resinous places must be covered with shellac varnish fore the priming coat is put on. Pitchy woods, such as southern yellow pine d cypress, do not readily absorb oil, and turpentine should be substituted for rt of the oil. Red lead is successfully used as a primer (2 parts to 1 of white d) on such woods; this is the standard practice in England, and is better than use of all white lead.

Outside Painting. The priming coat having largely been absorbed by the od, a second and third coat of paint are to be applied. The most common nt used on houses is white lead. This is commonly sold as paste white lead, taining 8% of oil; 100 lb of this is equal to 2.8 gal in volume, and is commonly xed with 3½ gal of raw linseed-oil, 1 qt of turpentine and 1 pt of drier to make gal of paint for the second coat; or with 4 gal of oil, 1 pt of turpentine and 1 of drier for the finishing coat. If white zinc is used, 9½ lb of dry zinc oxide 5.7 lb of oil make 1 gal of paint; to this, turpentine and drier should also added. White lead, after about a year, begins to CHALK, that is, its surface omes dry and chalky; this does not indicate failure, however, and it makes a d surface for repainting. Finely reticulated checking, not extending through film, occurs later, and when sufficiently marked indicates need of repainting. any paint, when cracks begin to extend through to the wood, repainting is ed for; these cracks occur sooner on pitchy woods. White zinc, if used alone outside (not inside) work, is very hard and tends to peel off. MIXED PAINTS epared proprietary paints) generally contain zinc mixed with either white l or some of the pigments based on basic lead sulphate, and some auxiliary

Table of Board-Measure

For explanation, see page 1559

Length in feet	Sectional area in square inches														
	4 ft	4 in	6 ft*	8 ft	8 in	10 ft	10 in	12 ft*	14 ft	14 in	16 ft	16 in	18 ft*	20 ft	20 in
6	2	0	3	4	0	5	0	6	7	0	8	0	9	10	[illegible]
8	2	8	4	5	4	6	8	8	9	4	10	8	12	13	[illegible]
10	3	4	5	6	8	8	4	10	11	8	13	4	15	16	[illegible]
12	4	0	6	8	0	10	0	12	14	0	16	0	18	20	[illegible]
14	4	8	7	9	4	11	8	14	16	4	18	8	21	23	[illegible]
16	5	4	8	10	8	13	4	16	18	8	21	4	24	26	[illegible]
18	6	0	9	12	0	15	0	18	21	0	24	0	27	30	[illegible]
20	6	8	10	13	4	16	8	20	23	4	26	8	30	33	[illegible]
22	7	4	11	14	8	18	4	22	25	8	29	4	33	36	[illegible]
24	8	0	12	16	0	20	0	24	28	0	32	0	36	40	[illegible]
26	8	8	13	17	4	21	8	26	30	4	34	8	39	43	[illegible]
28	9	4	14	18	8	23	4	28	32	8	37	4	42	46	[illegible]
30	10	0	15	20	0	25	0	30	35	0	40	0	45	50	[illegible]
32	10	8	16	21	4	26	8	32	37	4	42	8	48	53	[illegible]
34	11	4	17	22	8	28	4	34	39	8	45	4	51	56	[illegible]
36	12	0	18	24	0	30	0	36	42	0	48	0	54	60	[illegible]
38	12	8	19	25	4	31	8	38	44	4	50	8	57	63	[illegible]
40	13	4	20	26	8	33	4	40	46	8	53	4	60	66	[illegible]
42	14	0	21	28	0	35	0	42	49	0	56	0	63	70	[illegible]

Length in feet	Sectional area in square inches												
	24 ft*	28 ft	28 in	30 ft*	32 ft	32 in	35 ft	35 in	36 ft*	40 ft	40 in	42 ft*	48 ft*
6	12	14	0	15	16	0	17	6	18	20	0	21	24
8	16	18	8	20	21	4	23	4	24	26	8	28	32
10	20	23	4	25	26	8	29	2	30	33	4	35	40
12	24	28	0	30	32	0	35	0	36	40	0	42	48
14	28	32	8	35	37	4	40	10	42	46	8	49	56
16	32	37	4	40	42	8	46	8	48	53	4	56	64
18	36	42	0	45	48	0	52	6	54	60	0	63	72
20	40	46	8	50	53	4	58	4	60	66	8	70	80
22	44	51	4	55	58	8	64	2	66	73	4	77	88
24	48	56	0	60	64	0	70	0	72	80	0	84	96
26	52	60	8	65	69	4	75	10	78	86	8	91	104
28	56	65	4	70	74	8	81	8	84	93	4	98	112
30	60	70	0	75	80	0	87	6	90	100	0	105	120
32	64	74	8	80	85	4	93	4	96	106	8	112	128
34	68	79	4	85	90	8	99	2	102	113	4	119	136
36	72	84	0	90	96	0	105	0	108	120	0	126	144
38	76	88	8	95	101	4	110	10	114	126	8	133	152
40	80	93	4	100	106	8	116	8	120	133	4	140	160
42	84	98	0	105	112	0	122	6	126	140	0	147	168

* The measurements in these columns come out in even feet.

Table of Board-Measure (Continued)

For explanation, see page 1559

Length in feet	Sectional area in square inches: 56 ft	56 in	60 ft *	64 ft	64 in	72 ft *	80 ft	80 in	84 ft *	96 ft *	100 ft	100 in	112 ft	112 in
4	18	8	20	21	4	24	26	8	28	32	33	4	37	4
6	28	0	30	32	0	36	40	0	42	48	50	0	56	0
8	37	4	40	42	8	48	53	4	56	64	66	8	74	8
10	46	8	50	53	4	60	66	8	70	80	83	4	93	4
12	56	0	60	64	0	72	80	0	84	96	100	0	112	0
14	65	4	70	74	8	84	93	4	98	112	116	8	130	8
16	74	8	80	85	4	96	106	8	112	128	133	4	149	4
18	84	0	90	96	0	108	120	0	126	144	150	0	168	0
20	93	4	100	106	8	120	133	4	140	160	166	8	186	8
22	102	8	110	117	4	132	146	8	154	176	183	4	205	4
24	112	0	120	128	0	144	160	0	168	192	200	0	224	0
26	121	4	130	138	8	156	173	4	182	208	216	8	242	8
28	130	8	140	149	4	168	186	8	196	224	233	4	261	4
30	140	0	150	160	0	180	200	0	210	240	250	0	280	0
32	149	4	160	170	8	192	213	4	224	256	266	8	298	8
34	158	8	170	181	4	204	226	8	238	272	283	4	317	4
36	168	0	180	192	0	216	240	0	252	288	300	0	336	0
38	177	4	190	202	8	228	253	4	266	304	316	8	354	8
40	186	8	200	213	4	240	266	8	280	320	333	4	373	4
42	196	0	210	224	0	252	280	0	294	336	350	0	392	0
44	205	4	220	234	8	264	293	4	308	352	366	8	410	8
46	214	8	230	245	4	276	306	8	322	368	383	4	429	4
48	224	0	240	256	0	288	320	0	336	384	400	0	448	0
50	233	4	250	266	8	300	333	4	350	400	416	8	466	8
52	242	8	260	277	4	312	346	8	364	416	433	4	485	4
54	252	0	270	288	0	324	360	0	378	432	450	0	504	0
56	261	4	280	298	8	336	373	4	392	448	466	8	522	8
58	270	8	290	309	4	348	386	8	406	464	483	4	541	4
60	280	0	300	320	0	360	400	0	420	480	500	0	560	0
62	289	4	310	330	8	372	413	4	434	496	516	8	578	8
64	298	8	320	341	4	384	426	8	448	512	533	4	597	4
66	308	0	330	352	0	396	440	0	462	528	550	0	616	0
68	317	4	340	362	8	408	453	4	476	544	566	8	634	8
70	326	8	350	373	4	420	466	8	490	560	583	4	653	4
72	336	0	360	384	0	432	480	0	504	576	600	0	672	0
74	345	4	370	394	8	444	493	4	518	592	616	8	690	8
76	354	8	380	405	4	456	506	8	532	608	633	4	709	4
78	364	0	390	416	0	468	520	0	546	624	650	0	728	0
80	373	4	400	426	8	480	533	4	560	640	666	8	746	8
82	382	8	410	437	4	492	546	8	574	656	683	4	765	4
84	392	0	420	448	0	504	560	0	588	672	700	0	784	0

* The measurements in these columns come out in even feet.

a fire. It is especially suitable for cleaning out moldings and all irregular sur-faces from which the varnish may then be removed with stiff brushes, if it is not convenient to use scrapers. It is especially desirable to have floors occasionally cleaned in this way; but if a house has been varnished originally with a first-class varnish it may be necessary only to wash it thoroughly and then apply another coat of varnish. Smoke and dirt may often be thoroughly removed from ceilings with the crumbs of fresh bread, where washing would not be desirable. A 10% solution of carbonate of soda (sal soda) in hot water may be used to remove old floor-wax.

The Painting of Structural Steel. Steel being usually more perishable than wood, as well as more expensive, and used for service where its strength is essential to the stability of the structure, its protection from corrosion by painting is of much importance. It must first of all be recognized that the precautions always taken in painting wood, to secure a clean surface for the paint, must not be omitted with steel. Mud and dirt must first be removed from the steel; then it must be examined for rust, and any rust-spots must be thoroughly cleaned. Loose scale may be removed with wire brushes, but thick and closely adherent rust must be removed with steel scrapers, or with hammer and chisel if necessary. No doubt the best way to clean steel is to use the sand-blast, but it is not available for much architectural work. In any case much care must be taken to obtain a clean surface. On wood the priming coat sinks into the wood and forms a perfect bond between it and the succeeding coats; but on metal no such thing is possible and it is a case of simple adhesion, which demands a clean surface for efficient results. The paint for structural metal should be tough and elastic, and to as great a degree as possible it should be water-proof. Less than two coats should never be applied, and three are better. Paint is always thin on edges and angles, and also on bolt and rivet-heads; it is therefore good practice, after the first full coat, to apply a partial or striping coat, covering the angles and edges and the surface for at least 1 in back from the edges, and covering all bolt-heads and rivet-heads. After this striping coat has become dry, the second full coat is applied, and it may then be assumed that the whole surface has received two full coats. At least a week should elapse between coats. In designing the steelwork, all cavities which may be filled with rain during erection should be properly drained; and during erection all small cavities should be filled with cement, and all contact-surfaces thickly painted.

Kinds of Paint for Structural Steel. RED LEAD is more generally used than anything else as a paint for structural steel. It is a "true red lead" (Pb_3O_4), usually made from litharge (PbO), and frequently containing from 10 to 20% of the latter. If it contains much litharge, it rapidly thickens when mixed with oil and finally hardens; this makes it a paint difficult to apply. If, however, the material from which it is made is reduced to a sufficiently fine powder before it is oxidized an almost completely oxidized red lead is produced, which is as easily worked as white lead, and better in every respect. The requirements of the government of the United States have for years called for red lead of not less than 94% of "true red lead" (Pb_3O_4), and the Navy Department, as well as several large railway companies, is now using large amounts of red lead which has not less than 98% of "true red lead." It may now be obtained in paste-form, similar to white lead and containing about 6½% of raw linseed-oil. 33 lb of red lead (dry pigment) to 1 gal of oil is the maximum and this is especially suitable for hydraulic work; 28 lb to 1 gal of oil (containing 20 lb of pigment in a gallon of paint) is more common; while 25 lb to a gallon of oil is a common requirement for railroad-specifications. Finely ground GRAPHITE in linseed-oil is a favorite paint for metal; it flows well, is easily applied, less expensive than red lead, and if well

made gives excellent results. Graphite is sometimes mixed with lampblack, probably with advantage. Boneblack is also an important ingredient of CARBON PAINTS. Formerly oxide of iron in linseed-oil was used more than all other paints for this purpose; but while many engineers still like it, its use has very greatly diminished. ASPHALTUM has been used and is still used, as a varnish either alone or in combination, and some of these asphaltic preparations are fairly satisfactory. The fact is, that a really competent paint-manufacturer can make a reasonably good paint out of any of these, and if the paint is carefully applied the results will be satisfactory. There are great differences in painters. In regard to the surface of structural steel covered by a gallon of paint, there is a great difference of opinion among experts. Some say from 300 to 400 sq ft, others 1000 or 1200 sq ft. The truth is that any paint may be brushed out into an exceedingly thin film by a skilled workman, while ordinary usage results in a film at least twice as thick. The general opinion is that it is not wise to estimate more than 400 sq ft to the gallon for one coat. Varnish-paints cover less than oil-paints, but if well made they are very durable.

Painting on Cement and Concrete. Cement and concrete-work are difficult to paint, because they are strongly alkaline and even caustic when new. Work in these materials should be allowed to stand a year or two if possible before it is painted; then it may be painted with any ordinary paint. A practice which has been highly recommended is to wash the surface, repeatedly if possible, with a strong solution of zinc sulphate, the sulphuric acid uniting with the free lime and the zinc being left in the pores as an oxide or hydrate. Some preparations for this purpose are on the market; and while some are probably good, others are to be distrusted. The best way is to allow the surface to age, if this is at all possible.

WINDOW-GLASS AND GLAZING *

Glazing. The glazing of windows originally belonged to the painter's trade, and when glass is broken, it is still customary to go to a painter to have it replaced; but custom has so changed in some parts of the country, that when new windows are to be glazed, the work is sometimes done at the mill or factory where the sashes are made, sometimes by the local glass-jobber in the town where the building is being erected, and again, in other localities, the glazing of new buildings is still done by the painter. COMMON WINDOW-GLASS is usually set with putty and secured with triangular pieces of zinc called GLAZIERS' POINTS, driven into the wood over the glass and covered with putty. In the best work, a thin layer of putty is first put in the rebate of the sash and the glass is then placed on it and pushed down to a solid bearing. This is called BACK-PUTTYING. The points are then driven about 8 or 10 in apart and the putty applied over the glass and points so as to fill the rebate. Outside windows should always be glazed on the outside of the sash. Common window-glass has a slight bend in it, the result of its original cylindrical shape; it should be glazed, therefore, with the convex side out, as this reduces to a minimum the effects of the waviness when looking through it either from the outside or inside. Plate glass, in window-sashes and door-lights, should be back-puttied and secured by wooden beads.

Leaded Glass. It was formerly a common practice for architects to name in the specifications a certain sum of money to be allowed by the carpenter for the leaded glass and to be expended under the direction of the architect. Where

* Condensed from article on Window-Glass and Glazing by Professor Thomas Nolan in Building Construction and Superintendence, Part II, Carpenters' Work, by F. E. Kidder.

clear glass was used, the pattern was sometimes shown on the drawings and the glass was specified in the same manner as any other work. When colored glass was to be used, it was customary to make a definite allowance and then to entrust the work to a good art-glass manufacturer. But leaded glass should be designed, furnished and put in place by those who are entirely familiar with its manufacture and its limitations; the purchase of the same should be left entirely in the hands of the owner; and no specification as to its price or make should be used by the architect. The colored-glass windows should show as much individual artistic taste as any other picture or decoration used in the building. The cheap and inartistic leaded glass is fast becoming a thing of the past and owners are confining themselves to purely works of art placed in some appropriate location in the building.

Sheet Glass. General Description. Common window-glass is technically known as SHEET GLASS or CYLINDER GLASS. "It is made by the workmen dipping a tube with an enlarged end in the molten glass or METAL until from 7 to 10 lb are gathered up. Then it is blown out slightly by the workman, taken on a blowing-tube and still further blown and manipulated, until a cylinder about 15 in in diameter and 60 in long is formed. This cylinder has the two ends trimmed off and is then cut longitudinally and gradually warmed. It is then placed on a large flat stone supported by a carriage, where it is heated until it softens sufficiently to open out flat; the carriage is then pushed into the annealing-chamber and the sheet taken off." About the year 1910, sheet glass blown by machinery, utilizing compressed air, was perfected, and the result has been a gradual decrease in its cost. The cylinder blown by compressed air is split open and flattened out in just the same manner and by the same process as in the mouth-blown cylinder.

Grades and Qualities of Sheet Glass. Sheet glass is graded as DOUBLE-THICK or SINGLE-THICK, and each thickness is further divided into three qualities, FIRST, SECOND, or THIRD, according to its relative freedom from defects. The price varies according to the strength and quality. It should be remembered that sheet glass is always wavy, the result of the flattening of the cylinder. Many suppose that by designating sheet glass, CRYSTAL-SHEET GLASS, SELECTED-SHEET GLASS, or SHEET GLASS FREE FROM WAVES AND IMPERFECTIONS, a sheet glass free from waves and blemishes can be obtained. The terms and names do not change the nature of this glass, which still remains sheet glass, characterized by the defects inherent in the method by which it is manufactured. To obtain a thin glass, free from waviness, plate glass, ⅛ in thick, sometimes known as CRYSTAL PLATE, or plate glass 3⁄16 in thick, must be specified. Since the improvement in the manufacture of window-glass in this country, scarcely any sheet glass is now imported for glazing purposes. A small amount of Belgian sheet glass is brought to this country and used along the Atlantic seaboard for picture-framing. The low prices of the American sheet glass, and its excellent quality, have practically forced imported sheet glass out of the market. All common sheet glass, without regard to quality, is graded according to thickness, as SINGLE-THICK or DOUBLE-THICK. The thickness of the double-thick glass is a scant ⅛ in while that of the single-thick averages about 1⁄12 in. It is customary to use the double thickness for sheet glass over 24 in in width. The best quality of sheet glass is specified as AA, the second as A and the third as B.

Sizes of Sheet Glass. The regular stock-sizes vary by inches from 6 to 16 in in width. Above that they vary by even inches up to 60 in in width and 70 in in length for double thickness, and up to 30 by 50 in for single thickness.

Cost of Sheet Glass. The prices for sheet glass, as for all other clear glass, vary wi[illegible] [illegible]ze, strength and quality. Prices are determined by a schedule

or price-list,* giving the price for each size, in both thicknesses, and all qualities; and from these prices a very large discount is allowed. Fluctuations in prices are regulated by the discount, the list usually remaining unchanged for a number of years. The price-list prevailing in 1913 was in use from October 1, 1903. The only way to ascertain the price of a light of glass of a given size is to find it from the price-list, from which the discount, quoted by the glass-dealer, must be deducted. For the benefit of the Pacific Coast trade there is a Western Glass List † which differs somewhat from the Eastern list. The list is for sheet glass, the plate glass lists being the same in the East and West. The price per square foot increases rapidly as the size of the pane increases, so that it is much cheaper to divide a large window into eight or twelve lights than into two lights. Compared with the cost of the building, however, the glass is a small item and in the better classes of buildings each sash is usually glazed with a single light of glass. In factories, workshops, etc., where there is usually a large amount of glass-surface, the size of the lights is not of so much importance, while the saving by using small lights is quite an item; hence twelve-light and even sixteen-light windows are generally used in such buildings. The following table shows quite clearly the relative cost (1913) per square foot of different-sized panes of American glass, the prices given being an average at that time for the whole country.

Comparative Cost (1913) of American Sheet Glass per Square Foot, Based Upon a Discount of 90 and 20 Per Cent on the List of October 1, 1903 ‡

Grades	Sizes of lights in inches						
	10×12	15×20	24×34	30×36	36×40	40×60	60×70
	Prices in cents per square foot						
Double strength:							
First quality......	7.0	8.3	9.4	10.0	10.8	14.0	29.2
Second quality....	6.0	7.3	8.3	9.0	10.0	14.4	27.0
Single strength:							
First quality......	5.0	4.8	6.4	6.8	......	......	......
Second quality....	4.3	4.5	5.6	6.0	......	......	......

Crystal-Sheet Glass, 26-Ounce. This glass is made by the cylinder-process, it is a little thicker than the ordinary double-strength glass. It is probably the best glass made, next to plate glass, but owing to the method of its manufacture is necessarily characterized by a wavy appearance. If good glass is required for first-class residences, hotels, office-buildings, etc., polished plate glass should be used. The latter invariably gives satisfaction, while sheet glass, no matter of what thickness, is usually disappointing in its appearance.

Defects of Sheet Glass. All sheet glass, when looked upon from the outside, has a wavy, watery appearance, like the surface of a lake slightly agitated by

* The price-lists of glass have been omitted as they can readily be obtained from the glass-dealers in any city. Such lists are not of much service unless they are complete; and the full lists are too long to be inserted in a condensed handbook.

† This list, with discounts from the prices given, may be obtained from the W. P. Fuller Company, San Francisco, Cal.

‡ Much valuable information in regard to Window-Glass and Glazing was furnished by Mr. S. C. Gilmore of the Hires-Turner Glass Company. Philadelphia, Pa.

the wind; and when the sunshine falls upon it the irregularity of the surface is greatly emphasized. This characteristic of sheet glass is due to its being made in the shape of a cylinder and then stretched or flattened out into a sheet, and it cannot be wholly avoided. Besides this universal defect, the cheaper grades are often STRINGY, BLISTERY, SULPHURED, SMOKED, or STAINED; so that, in looking through the glass, objects seen at a distance are deformed and distorted.

Plate Glass. General Description. Plate glass is commonly known as POLISHED PLATE GLASS because its surface is finely polished and thus made clear and transparent. It is more largely used every year for windows of fine residences, hotels and office-buildings, where transparency is desired from the inside and an elegant appearance required on the outside. The process of manufacture of plate glass is entirely different from that of sheet glass. In making plate glass the metal, which is prepared with great care, is melted in large pots and then cast on a perfectly flat cast-iron table. "The width and thickness of the plate is determined by means of metal strips called GUNS, which are fastened on, and on which a heavy, metal roller travels. The ends of the guns are tapered so that when the roller is at one extremity, it and the guns form three sides of a shallow, rectangular dish. The molten metal is poured on and the roller passed along slowly, forcing the metal in front of it and rolling out the sheet." The sheet is then annealed and forms what is known as ROUGH PLATE, which is used for vault-lights, skylights, floor-lights and the like. "For polished plate the rough plate is carefully examined for flaws, which are cut out, leaving the largest-sized sheet practicable. The plate is then fastened to a revolving table by means of plaster of Paris, and two heavy shoes, shod with cast iron, are mounted over it. The table is then revolved and sand and water fed onto the surface; the shoes revolve also, going over all parts of the plate and grinding it down to a true plane. Emery-powder is then fed on, in successive degrees of fineness until the plate is made absolutely smooth and all grit removed. After this, new rubbers, shod with very fine felt, are put on and liquid rouge is added for the polishing. When one side is completed the other side is similarly treated, the plate losing about 40% in weight by the operation."

Qualities of Polished Plate Glass. For glazing purposes there is but one quality of plate glass on the market. The best of this is selected for manufacturing mirrors. At one time, plate glass was extensively imported, but the gradually improving methods of the American manufacturers, as well as the great cheapening of the process, have practically eliminated imported plate glass from the market. The American plate glass is equal in every respect to that which was imported. The usual thickness of polished plate glass is from ¼ to 5/16 in, but it can be made thinner than this; and when required for residence-windows or car-windows, may be obtained in 3/16 or ⅛-in thicknesses. It is manufactured from the same thickness of rough plate used for the ordinary thicknesses, but is ground down thinner and, owing to the additional cost of grinding, as well as to the risk, is more expensive than glass of the ordinary thickness.

Cost* of Polished Plate Glass. The cost of plate glass of ordinary thickness varies with the size of the lights. The net price of polished plate glass (1913) glazing quality, was about 45 cts ($0.45) per sq ft, for sizes of not more than 10 sq ft per plate, 50 cts ($0.50) per sq ft for sizes containing from 10 to 50 sq ft per plate, and 65 cts ($0.65) per sq ft for sizes containing not more than 120 sq ft per plate. For larger sizes the price increased rapidly up to $2.00 per sq ft. The price, however, can be accurately determined only by means of a price-list and discount. The price-list in use (1913) was introduced in March, 1910, and the

* These are pre-war prices and the data are retained for purposes of comparison.

discount was about 90%. Plate glass 5/16 in thick costs 15% more than glass of the regular thickness on account of the extra expense of grinding it down. Plate glass 1/8 in thick costs from 25 to 40% more than glass of the regular thickness.

Sizes of Polished Plate Glass. Plate glass is cut into stock sizes, varying by even numbers from 6 by 6 in up to 144 by 240 in, or 138 by 260 in.

Comparative Cost of Different Kinds of Window-Glass. The following table gives an idea of the comparative cost of the different kinds and qualities of glass used in this country for glazing. The prices for the sizes are the 1914, net, average prices. The first column of the table gives the kinds of glass, and includes both the American plate and the American sheet glass. The other columns of the table give the sizes of the different lights in inches.

Comparative Cost of Different Kinds of Window Glass *

Kinds of glass	Sizes of lights in inches			
	24×32	30×36	36×40	48×60
American Plate Glass				
Glazing-quality	$2.35	$3.38	$4.60	$9.80
Crystal-sheet glass, 26-oz	1.00	1.54	2.34	6.66
American Sheet Glass				
Double-strength, first quality	0.54	0.83	1.25	3.55
Double-strength, second quality	0.47	0.73	1.13	3.30
Single-strength, first quality	0.37	0.56	......	......
Single-strength, second quality	0.32	0.50	......	......

It will be seen from this table that the relative difference in the cost of plate and sheet glass decreases rapidly as the sizes of the lights increase. The prices in this table are based on the list of October 1, 1903, on a discount of 90% for plate glass, 90 and 20% for American sheet glass and 85% on AA double-thick for 26-oz crystal-sheet glass.

Wire-Glass. This is described in Chapter XXIII, page 821.

Figured Rolled Glass. This is a translucent or OBSCURED glass with a pattern stamped on one surface. As the molten metal is rolled out on the table, the design, cut into the table, imprints itself into the soft glass. This kind of glass has almost entirely supplanted the ordinary ground glass because of its greater cleanliness. There are several popular designs on the market, made by various manufacturers. Some of the designs in common use are known as MOSS, MAZE, COLONIAL, FLORENTINE, COBWEB, etc. This glass is usually made 1/8 in thick and in large sheets from 24 to 42 in wide and from 8 to 10 ft long. MAZE, FLORENTINE and COBWEB designs can be had either with or without the wire mesh in them. One important property of figured rolled glass is that of diffusing the light which passes through it. (See, also, pages 1453 and 1554.)

Pressed Prism-Plate Glass.† This is manufactured in different patterns and for different purposes and includes (1) Imperial Prism-Plate Ornamental Glass in five different patterns, (2) Imperial Prism-Plate Glass and (3) Imperial Skylight Prism Glass. The general description is as follows:

* These are pre-war prices and the data are retained for purposes of comparison.
† Manufactured by the Pressed Prism Plate Glass Company, Chicago, Ill. See, also, pages 1453 to 1456.

(1) **Imperial Prism-Plate Ornamental Glass** is plate glass ground and polished on one side. It is manufactured in plates, 54 by 72 or 72 by 54 in, can be cut into smaller sizes, and is made in five different stock patterns. It is used in modern mercantile, office and public buildings for partitions, transoms, door-lights, vestibule doors, ornamental ceiling-lights, bank-windows and other street-windows, and in all places where semiobscurity and ornamental effect are desired. On account of its prismatic qualities it gives a strong diffusion of light for office-use where privacy is desired.

(2) Imperial Prism-Plate Glass. This is manufactured in large sheets, 54 by 72 or 72 by 54 in, and can be cut into smaller sizes. It is made in several different angles in order to obtain the proper diffusion of light for varying conditions. It is a plate glass, ground and polished on one side. There are no wires or bars to collect dirt and retard the light and it is very easily cleaned. It is used in the upper sashes of windows and in transoms, store-fronts, etc.

(3) Imperial Skylight Prism Glass. This is made in unit plates, 18 by 60 in, with a ½-in back, and conforms to the requirements of the Board of Fire Insurance Underwriters. It is used for skylights, roofs over areaways and in light-wells, etc. The possibility of leakage is lessened on account of the large-sized plates in which it may be obtained. These plates, however, can be cut into smaller sizes if required. It is particularly adapted for lighting the rear parts of stores and for railway-stations, sheds, etc.

Prism Glass, for glazing windows, skylights and sidewalk-lights, is now manufactured in a large number of forms in both prisms and sheets, and by several companies. The diffusing properties of several types are described on pages 1453 to 1456 under the subject of Illumination. This glass is made with sharp prisms which are glazed horizontally in the windows and by refracting the light throw it back horizontally into the rooms, adding very materially to the interior lighting. It is manufactured by several companies and can be procured from glass-jobbers in practically all the cities of the United States. (See, also, page 821.) Glass prisms for lighting are made of pieces of glass of standard dimensions, about 4 in square, with a smooth outer surface and an inner surface divided into a series of prisms. They are, in many cases, formed into plates by the process of electroglazing, the edges of the prism-lenses being welded together, so to speak, by a narrow line of copper which gives the desired stiffness and strength for use in large frames, and also an attractive appearance considered by some to be superior to ordinary leaded work. These prism-plates can be made in any desired size, but for very large surfaces two or more plates, divided by means of metal sash-bars, are generally used. (See, also, page 821.)

The commercial value of these prisms depends on that property of glass which causes what is known as REFRACTION. Prism-plates receive the light from the sky, not necessarily from the sun, and refract or turn it back into the room which is to be lighted. With an ordinary window the light from the sky, passing through the glass, strikes the floor at a point not very far distant from the window. As the color of the floor is usually dark, reflecting perhaps only one-tenth part of the light falling on it, the rear parts of the room receive only a small portion of the light which enters the window. For this reason it has been necessary to make very high stories for deep rooms, in order to light, even moderately, those parts which are at a distance from the window. When prisms are substituted for the common window-glass or plate glass, the rays of light as they enter the glass are refracted, and by employing prisms of the proper angle, the rays may be given almost any direction. Moreover, by utilizing different prisms in the same plate, some of the rays may be directed to the rear of the room while others are thrown so as to strike near the front. The prism-plates do not increase

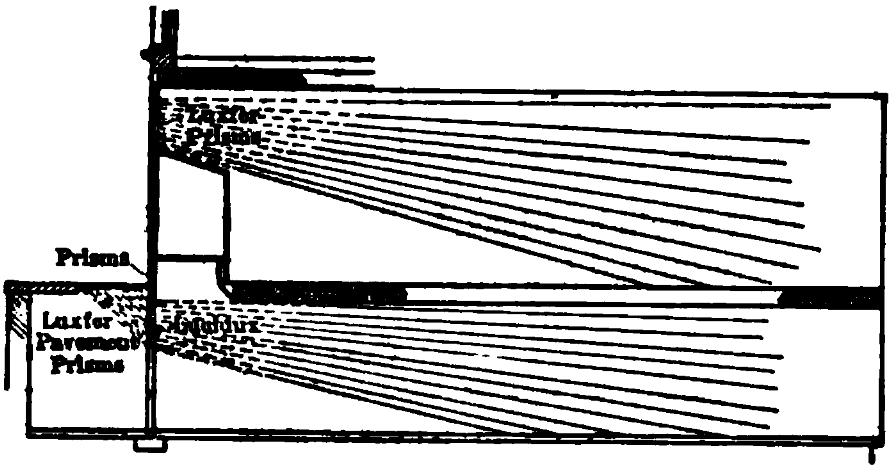
uantity of light entering the window, but simply redistribute it, directing it nto those portions of the room in which it is most needed. By thus changing he direction of light-rays a room with a low ceiling can be better lighted than vhen sheet or plate glass is used. To insure success in the lighting of interiors y means of prisms requires, however, a superior quality of glass, and careful cientific calculations and experiments, besides practical and attractive means f glazing and methods of installation. These requirements have been met by he several companies making these prisms and their products may be considered among the relatively new building materials. They have been very uccessfully applied to the lighting of dark rooms by daylight. The application f prisms to any particular building depends upon the surrounding conditions nd requirements, each case requiring some special treatment; but in a general ay the various appliances used in the installations may be divided into four asses as follows:

(1) Vertical Plates, which are set directly in the sashes in place of the ordinary indow-glass. They are commonly used for the transom-lights of store-windows d the upper sashes of double-hung windows. They may also fill the entire indow.

(2) Foriluxes, which are vertical prism-plates set in independent frames and aced in window-openings substantially flush with the face of the wall.

(3) Canopies, which are external prism-plates in independent frames, placed er window-openings and set at an angle with the vertical, a position similar to at of an ordinary awning.

(4) Pavement-Prisms, which are set in iron frames in the pavements or sidelks, in place of the ordinary bull's-eye lights. In connection with the pavement-prisms, when a ll-lighted basement is sired, vertical plates of isms, hung below and posite the pavementhts, are often used. ese hanging, vertical tes receive the light m the pavementsms, and again changits direction, project horizontally into the ement. This feature illustrated in the



Refraction and Transmission of Light by Prisms

ure here given, reproduced through the courtesy of the Luxfer Prism mpany.

The canopies may be made either stationary or adjustable and may be emyed in a variety of ways, combining the useful with the ornamental. The ging, vertical plates lend themselves to a highly decorative treatment. In h the fixed and hanging vertical plates the prisms may be arranged to proe ornamental effects, and designs may be inwrought on the face of the prismtes to correspond with the designs worked into the surfaces of the building with the style of the entire façade. The prism-plates weigh no more, and n less, than plate glass of the same size, while they are much stronger in sting wind-pressure, the action of hail and the impact of flying fragments. ough transmitting a very large amount of light, these prism-plates are not sparent in the ordinary sense, and may thus be used as screens to hide unactive views or to prevent persons looking either in or out of a window. At

the same time a maximum quantity of light is admitted. The prism-pl[ates] owing to the stiff, durable manner in which they are united by the elect[ro-] glazing process, serve, also, as a fire-retardant or as a partial substitute for [the] ordinary iron fire-shutters. The copper glazing forms, as it were, a contin[uous] rivet, which holds the individual prism-lights together, even after they h[ave] become badly cracked by the action of fire and water. The details of the va[ri-] ous makes of prisms are too complicated to be set forth in a few pages, but th[ey] are well described in the various handbooks and catalogues published by [the] different manufacturers. From a commercial point of view the special a[d-] vantages of these systems of interior lighting are manifold. They transf[orm] rooms, particularly basements, otherwise too dark for occupancy, into inco[me-] producing spaces; in many buildings they do away with the use of light-sha[fts,] thus saving a large amount of valuable floor-space; and in all large or de[ep] rooms they effect a great saving in artificial lighting. Once installed, the[re is] no cost for maintenance. The extent to which these prisms have been used b[y] architects, in both new and old buildings, shows that they have had a deci[ded] influence upon commercial architecture.

Glass for Skylights. General Description. The glass ordinarily used n[ow] for skylights is either rough or ribbed skylight-glass, and since the great cheap[en-] ing in the process of manufacturing glass with wire mesh in it, wire-glass, also, [is] being largely used for this purpose. The sizes used depend largely upon t[he] pitch of the skylight, small sizes being more desirable when the pitch is sl[ight.] The weight of rough or ribbed glass, with or without wire mesh, is approximat[ely] as follows:

Weight of Rough or Ribbed Glass

Thickness in inches........	1/8	3/16	1/4	3/8	1/2	5/8	3/4	1
Weight in pounds..........	2	2½	3½	5	7	8½	10	[illegible]

Cost of Skylight-Glass.* The different kinds of skylight-glass in small qua[n-] tities were quoted (1914) about as follows:

Cost of Skylight-Glass

Kinds of glass	Cost
Rough or ribbed skylight-glass, 1/8-in..............................	6 cts per sq[.]
Rough or ribbed skylight-glass, 3/16-in..............................	8 cts per sq[.]
Rough or ribbed skylight-glass, 1/4-in..............................	12 cts per sq[.]
Rough or ribbed wire-glass, 1/4-in..............................	16 cts per sq[.]
Maze, Cobweb, or Florentine wire-glass..............................	20 cts per sq[.]
Sheet prism glass..............................	20 cts per sq[.]

Glass for Mirrors. Mirrors are made by silvering one side of a shee[t of] polished plate glass. This is the only kind of glass suitable for making mirr[ors] because, unless the surface of glass is polished, the reflection is distorted. [A] generation ago, mirrors were made by the old-style process of pressing the g[lass] by means of heavy weights onto mercury, backed by tinfoil, the affinity of [mer-] cury for tin forming an amalgam which protected the back of the mirrors [and] gave the reflection. This was a very slow and expensive process. During [the] twenty-five years prior to 1913, practically all of the mirrors made [were] manufactured by what is known as the PATENT-BACK process, in which [...]

* The prices have materially advanced and as they change from year to year, the m[an-]ufacturers' lists must be consulted.

silver is precipitated in a film over the surface of the glass, thus giving it the operty of reflecting. This film is afterward covered and protected by shellac, rnish and paint. This modern method of manufacture has made it possible supply mirrors in considerably less time, and at a very much lower cost, than hen manufactured by the old-fashioned MERCURY-BACK process. There are any who claim that in spite of modern processes of manufacture, the old ethod produced the best results as far as durability is concerned. This is idenced by the following statement inserted by Mr. Kidder in the preceding ltions of the Pocket-Book: "There are two kinds of mirrors on the market, e the old time reliable mercury-back mirror, the other the nitrate of silver, or at is better known to the trade as the patent-back mirror. The latter is now d has, in recent years, been most extensively sold as a substitute for the former. the manufacture of mercury-back mirrors no chemicals are used, only two tals, mercury and tin-foil. The affinity of mercury for tin forms an amalgam pervious to and not affected by the atmosphere. A mercury-back mirror is iversally considered to be the only durable and permanent mirror. A nitrate-silver or patent-back mirror is produced by the precipitation of a chemical ution of nitrate of silver and other media on the surface of the glass, to which added one coat of shellac varnish overlaid with one or more coats of paint. is mirror, irrespective of the quality of the glass from which it is made, will adily deteriorate from the date of its manufacture to that of its final collapse, ich may occur at any time from a few months, but certainly within a few rs."

MEMORANDA ON ROOFING

Shingles.* The best shingles are those made from cypress, cedar, redwood, ite and yellow pine and spruce, in the order mentioned. Redwood, while haps not quite as durable as cypress, is less inflammable; sawed pine shingles inferior to cedar, and spruce shingles are not suitable for good work.

Number and Weight of Cedar and Pine Shingles Per Square of One Hundred Square Feet

ngth, in	Assumed width, in	Weather or gauge, in	Number of shingles per square †	Weight per square		Number of nails per square	Weight of nails per square, lb
				Cedar, lb	Pine, lb		
14	4	4	900	210	233	1 800	4.50
15	4	4½	800	200	222	1 600	4.00
16	4	5	720	192	213	1 440	3.60
18	4	5½	655	197	218	1 310	3.28
20	4	6	600	200	222	1 200	3.00
22	4	6½	554	203	226	1 108	2.77
24	4	7	515	206	229	1 030	2.58

izes of Shingles. Cedar and redwood shingles as commonly sawed are 20 in ngth, and cypress shingles usually from 20 to 24 in long, the longer ones allow-

* For more complete information see Kidder's Building Construction and Superintene, Part II, Carpenters' Work, pages 321 to 325.

† To allow for waste, add from 6 to 10%, the greater allowance being for the shorter les.

ing a greater exposure to the weather. Redwood shingles and the cedar shin from the States of Washington and Oregon, which States furnish most of shingles used west of the Mississippi, are 5/16 and 9/16 in thick at the b cypress shingles are usually sawed thicker. Those used in Boston are 3/16 thick. Ordinary roofing-shingles are of random widths, varying from 2½ 14 and sometimes 16 in. They are put up in bundles, usually four bundles the thousand. A THOUSAND common shingles means the equivalent of 1 0 shingles 4 in wide.

Dimension-Shingles are sawed to uniform width, either 4, 5, or 6 in. Dim sion-shingles with the butt sawed to various patterns are also carried stock.

On hip-roofs, or for four valleys, add 5% for cutting. On irregular roofs wi dormer-windows, add 10%. It is claimed that redwood shingles will go farth than cedar shingles. With a rise to the roof of from 8 to 10 in to the foot, ced shingles, or any shingles 16 or 18 in in length, should be laid from 4 to 4¼ in the weather; with a rise from 10 to 12 in, from 4¼ to 4⅜ in to the weath and on steeper roofs they may be laid from 4½ to 5 in. Redwood shingles m be laid ½ in more to the weather. Some authorities allow slightly greater e posures for these lengths. Where the longer shingles are used the exposure the weather may be increased up to 7 in for the 24-in lengths. On walls ced shingles are commonly laid 5 in to the weather, and redwood shingles 6 in.

Labor. An average shingler should lay 1 500 shingles in 9 hours on pl work; on irregular roofs with dormers, 1 000 per 9 hours.

Nails. It requires from 3½ to 4½ lb of threepenny or from 3½ to 6½ lb o fourpenny nails to 1 000 shingles, depending upon width and length of shingles.

Slate Roofs

Characteristics of Good Slate. A good slate should be both hard and toug If the slate is too soft, however, the nail-holes will become enlarged and the sla will become loose. If it is too brittle the slate will fly to pieces in the process squaring and holing and will be easily broken on the roof. "A good slate shoul give out a sharp metallic ring when struck with the knuckles; should not split under the slater's axe; should be easily HOLED without danger of fracture, a should not be tender or friable at the edges." The surface when freshly spl should have a bright metallic luster and be free from all loose flakes or dull s faces. Very few of the Vermont slates, however, have the metallic luster ribbons. Most slates contain ribbons or seams which traverse the slate approximately parallel directions. Slates containing soft ribbons are inferi and should not be used in good work.

Color. The color of slates varies from dark blue, bluish black, and purple gray and green. There are also a few quarries of red slate. The color of the slate does not appear to indicate the quality. All slate quarried in Maine black as is also that quarried in Virginia, while that quarried in Pennsylv and Maryland is also black but borders on dark blue and is advertised by firms as dark blue. Slate quarried in New York State is red, of various while that quarried in Vermont is of various colors, such as green, purple, v gated, etc. The red and dark colors were formerly considered the most tive but at the present time the greens are going on some of the largest finest of the new residences. Some slates are marked with bands or pa of a different color, and the dark-purple slates often have large spots of green on them. These spots do not as a rule affect the durability of the but they greatly detract from its appearance.

Grading of Slates. The Monson, Me., slates and Brownville, Me., slates re graded as follows: No. 1. Every sheet to be full 3/16 in thick, both ides smooth and all corners full and square. No pieces to be winding or varped.

No. 2. Thickness may vary from 1/8 to 1/4 in, all corners square, one side generlly smooth, one side generally rough, no badly warped slates.

The Bangor, Pa., slates are graded:

No. 1 Clear. A pure slate without any faults or blemishes.

No. 1 Ribbon. As well made as No. 1 Clear, except that it contains one or ore RIBBONS (a black band or streak across the slate), which, however, are high nough on the slate to be covered when laid, thus presenting a No. 1 roof.

No. 2 Ribbon. This contains several RIBBONS, some of which cannot be overed when laid.

No. 2 Clear. A slate without RIBBONS, made from rough beds.

Hard Beds. A clear Bangor slate, not quite as smooth as No. 1 Clear, but uch better than No. 2 Clear.

Ordinary Bent Slate. A smooth slate similar to No. 1 Clear, but bent at a dius of about 12 ft.

Punching. Formerly nail-holes in slates were punched on the job; now, hower, slates are bored and countersunk at the quarry, when so ordered. Architts should always specify that the slates are to be bored and countersunk, as inching badly damages the slates.

Sizes. The sizes of slates range from 9 by 7 in to 24 by 14 in, there being some irty-seven different sizes; the more common sizes, however, are the following: e sizes of slates best adapted for plain roofs are the large wide slates, such as by 16 in, 18 by 12 in, 20 by 12 in, or 24 by 14 in. Slates from 8 by 16 to 10 20 in are popular sizes, 9 by 18-in slates being probably used oftener than ose of any other size. The 11 by 22 and 12 by 24-in slates are used principally very large high buildings. The lower grades of slate are used largely on wareuses and barns. The larger sizes make fewer joints in the roof, require fewer ils, and diminish the number of small pieces at hips and valleys. For roofs t up into small sections the smaller sizes, such as 14 by 7 in or 16 by 8 in, look e best.

Thickness. Slates vary in thickness from 1/8 to 3/8 in; 3/16 in is the usual thicks for ordinary sizes (see Grading of Slates in the preceding paragraphs). It of utmost importance for architects to specify the thickness of slates, either ly 3/16 in thick, or fully 1/4 in thick, to secure a strong and durable roof.

Laying. Slates are laid either on a board sheathing (rough, or tongued and ooved) covered with tarred or water-proof paper or felt, or on roofing-laths m 2 to 3 in wide and from 1 to 1 1/4 in thick, nailed to the rafters at distances art to suit the gauge of the slates. Each slate should lap the slate in the ond course below, 3 in. The slates are fastened with two threepenny or fourny nails, one near each upper corner. For slates 20 by 10 in or larger, fourny nails should be used. Copper, composition, tinned, or galvanized nails uld be used. Plain-iron nails are speedily weakened by rust, and they ak and allow the slates to be blown off. On iron roofs slates are often ced directly on small iron purlins spaced at suitable distances apart to receive m, and fastened with wire or special forms of fasteners. THE GAUGE of a te is the portion exposed to the weather, which should be one-half the reinder obtained by subtracting 3 in from the length of the slate. Roofs to be ered with slate should have a rise of not less than 6 in to the foot for 20-in or in slates, or 8 in for smaller sizes.

Elastic Cement. In first-class work, the top course of slate on the ridge, a slate for from 2 to 4 ft from all gutters and 1 ft each way from all valleys a hips, should be bedded in elastic cement.

Flashings. By FLASHINGS are meant pieces of tin, zinc, or copper laid slate and up against walls, chimneys, copings, etc.

Counterflashings are of lead or zinc, and are laid between the courses in bri and turned down over the flashings. In flashing against stonework, grooves reglets often have to be cut to receive the counterflashings.

Close and Open Valleys. A close valley is one in which the slates are mite and flashed in each course and laid in cement. In such valleys no metal can seen. Close valleys should only be used for pitches above 45°. An open val is one formed of sheets of copper or zinc 15 or 16 in wide, over which t slates are laid.

Old English Method of Laying Slates.* This method of laying slate i volves the use of different shades of colored slates in graduated courses a random widths beginning at the eaves, for example, with slates 28 in long and 1 in thick, and using the different thicknesses from 1¼ to ⅜ in, in shorter length in working upward on the roof. The use of this kind of work for roofs has i creased in recent years and the method possesses vast possibilities for car ing out architects' ideas for varied artistic effects. The slates are made wi rough-cut edges in all thicknesses from 3/16 to 1½ in, in a combination of vari shades carefully selected in such proportion as to produce the best possible h mony, when laid. As all of these colors and shades are unfading, the WEATHE effect is obtained at once and is permanent. These slates are made not only i usual sizes, but in the OLD-ENGLISH STYLE, to be laid in graduated courses of d ferent lengths and in random widths. The Old English color-combination ro ing-slates should be specified to secure the light-and-shadow effect, and it is of t utmost importance to specify the thickness desired, as the price is the same f all sizes, while the cost varies according to thickness. When graduated cour are desired, specifications should call for the number of courses to be laid in ea length and thickness beginning at the eaves-courses, where the thickest sla are used in the largest sizes, sometimes 30 or even 36 in in length, and worki upward on the roof with the shorter lengths and thinner slates to the ridges wh the smallest sizes and thinnest slates are used. To secure a rough effect at mi mum cost, specifications should call for Old English color-combination, all sla to be fully ¼ in thick with rough cut edges and graduated courses in sizes ra ing from 24 by 16 to 12 by 6 in, with nail-holes drilled and countersunk. T secure the best rough effect, specifications should call for eaves-courses not le than ¾ in thick, stating the thickness desired for the eaves, and the number courses desired in each length and thickness. Among the good specimens of t Old English style of roofing may be mentioned the buildings of Princeton U versity for the Graduate College, where different shades of unfading-gr slates are used in thicknesses running from 1¼ in at the eaves to ⅜ in at t ridge.

Measurement. Slates are sold by the SQUARE, by which is meant a suffici number of slates of any size to cover 100 sq ft of surface on a roof, with 3 in lap, over the head of those in the second course below. The square is also t basis on which the cost of laying is measured. "Eaves, hips, valleys, and c tings against walls or dormers are measured extra; 1 ft wide by their wh

* Full information in regard to the details of the slates for this purpose and the meth employed in laying them can be obtained from the various companies.

ngth, the extra charge being made for waste material and the increased labor quired in cutting and fitting. Openings less than 3 sq ft are not deducted, and ll cuttings around them are measured extra. Extra charges are also made for orders, figures, and any change of color of the work and for steeples, towers, and erpendicular surfaces."*

Cost.† The cost of slates varies with the size, color and quality. The prices iven in the following table were about the average in 1915 for blue-black slate, of o. 1 grade, loaded on the cars at the Pennsylvania quarry. The freight in ır-load lots of 60 squares or over to Philadelphia from Bethlehem, Pa., was 60 s per square, from Pennsylvania to Omaha, Neb., $2.60 and from Vermont, oout the same. It will be seen that slates of the MEDIUM sizes cost the most, id those of the larger and smaller sizes the least. Special prices are quoted for ecial sizes. The larger sizes make the cheapest roofs. Red slates cost from to 150% more than black slates. The green slates are more expensive than e black with the exception of the Maine and Peach Bottom varieties.

Number and Cost † of Slates, and Pounds of Nails to 100 Square Feet of Roof

3-inch Lap

Sizes of slates, in	Exposed when laid, in	Number to a square	Weights of galvanized nails, lb	oz	Cost per square at quarry
14×24	10½	98	4d 1	6	$4.50
12×24	10½	115	1	10	4.50
12×22	9½	126	1	12	4.75
11×22	9½	138	1	15	4.75
11×20	8½	155	2	0	5.25
10×20	8½	170	2	6	5.25
12×18	7½	160	3d 1	13	....
10×18	7½	192	2	3	5.25
9×18	7½	214	2	7	5.25
12×16	6½	185	2	2	....
10×16	6½	222	2	8	....
9×16	6½	247	3	0	5.25
8×16	6½	277	3	2	5.25
10×14	5½	262	3	0	....
8×14	5½	328	3	12	4.75
7×14	5½	375	4	4	4.75
8×12	4½	400	4	9	....
7×12	4½	457	5	3	4.25
6×12	4½	534	6	1	4.25

The cost of blue-black-slate roofs, complete, varies from $9 to $16 per square, pending on the class of work and remoteness from the quarries. The addinal cost of laying slate in elastic cement varies from $1.75 to $2.50 per square. experienced roofer will lay, on an average, 2½ squares of slate in 8 hours.

Weight. Slate roofing 3/16 in thick will weigh on the roof about 6½ lb per sq ft, d if ¼ in thick, 8¾ lb, the smaller sizes weighing the most on account of the . The actual weight of a square foot of slate ¼ in thick is 3.63 lb. A cubic

* The Building Trades Pocket-book.

† These prices have advanced and the manufacturers' lists must be consulted.

foot of Vermont slate weighs approximately 175 lb. The average shipping weight for No. 1, 3/16-in slates, is approximately 725 lb; for 1/4-in slates, 1 000 lb; for 1/2-in slates, 2 000 lb, etc.

Roofing-Tiles

General Notes on Roofing-Tiles. The term ROOFING-TILE is commonly understood to refer to exterior roof-covering made from clay in units of various shapes and laid with overlapping edges. Clay or terra-cotta roof-tiles have long been very largely used in Europe, where their cost is much less than in America. Since the year 1893 the advance here in the character and extent of roofing-tile has been marked and rapid. This material can now be had at much lower prices than formerly prevailed, and the result has been that thousands of squares of terra-cotta tiles have been placed on shops and factories which would under former conditions have been covered with slate or metal. Whether or not a tile roof is as durable and satisfactory as one of No. 1 slate is a much-disputed question. Mr. Kidder was of the opinion that, considering the quantities used, slates have given better satisfaction than tiles. A tile roof, however, is certainly more attractive than a slate roof, and it is generally held that there are many roofing-tiles on the market which if properly laid prove as tight and durable as slates. There are so many patterns of roofing-tiles that it is impossible here to enter into a description of them. Of the various patterns, those which interlock are considered from a practical standpoint, to make the most satisfactory roof.

Laying Roofing-Tiles. Roofing-tiles have been laid directly on a porous book tile or concrete base or on a sheathed surface over such base, or they have been fastened to stripping over the sheathing or wooden or steel purlins by means of copper wires. When thus fastened by wires, the joints were usually pointed on the under side after they were laid, to prevent the entrance of dust or dry snow. Tiles of the older patterns were nailed to the sheathing, but later on this method was superseded by the practice of fastening with copper wires from pierced lugs near the lower ends of the tiles. The best modern method, however, seems to be the one involving a solid continuous base for the roofing-tiles, whether or not purlins are used. "Such purlins should be filled in between either with book tiles or a concrete base and felt should be laid thereon. The book tiles, if used, should be of a porous quality. Instead of regarding the nailing of tiles as a defective method, we have returned to it as the only proper method of fastening tiles and have eliminated the stripping of sheathed roofs and the use of copper wires. Such methods would do in some portions of central Europe where the winds and other climatic conditions are not severe, but through a twenty-five years' experience in the varied climatic conditions of the United States, we have found that the nailing of tiles with copper nails is the only satisfactory method of application. We have also found that a roof should be sheathed and covered with a good asphaltum-felt to prevent wind-suction." * Roofing-tiles weigh from 750 to 1 200 lb per square of 100 sq ft.

Specifications for Tile Roofing

The following specification † contains valuable suggestions for the proper laying of tile roofs:

All pitched roofs shall be covered with (——) tiles with fittings suitable for

* Quoted by permission from data on roof-tiling, by the Ludowici-Celadon Company, Chicago, Ill.

† Prepared from data furnished by the Ludowici-Celedon Company, Chicago, Ill.

each pattern unless otherwise selected by the architect. The tiles as specified above are to be hard-burned, of red color, and in accordance with samples deposited in the office of the architect.

(1) Preparation of Roof. Before the roofer is sent for, the owner or general contractor is to construct the roofs in strict accordance with the plans, sheath the roofs TIGHT, have all chimneys and walls above the roof-line completed, have all vent-pipes put through the roofs, furnish all strips of required width used under hip-rolls, furnish all 1 by ⅞-in cant-strips used under the tiles at the eaves and have all the scaffolding ready for the roofers' use. The metal-contractor is to have all gutters in place on the roof (gutters, whether box, hanging or secret gutters, are to extend over the roof-sheathing and cant-strips, and run under the felt and tiles at least 8 in) and is to have in place, also, all valley-metal, the width of which is to be not less than 24 in, with both edges turned up ¼ in through the entire length of the valley. The valley-metal is to be fastened with clips and never nailed or punctured in any manner. The valley-metal is to be laid over one layer of felt running lengthwise the entire distance of the valley. The metal-contractor is to have in readiness all flashing-metal used alongside and in front of dormers, gables, skylights, towers and perpendicular walls, and around vent-pipes and chimneys, and is to place the same after the arrival of the tile-roofer and under his direction.

(2) Laying the Felt. After the roofs have thus been prepared to receive the felt and tiles, the tile-roofer is to cover the sheathing of the roofs with one thickness of asphalt roofing-felt weighing not less than 30 lb to the square, laying the same with a 2½-in lap and securing it in place by capped nails. The felt is to be laid parallel with the eaves, lapped over all valley-metal about 4 in and laid under all flashing-metal about 6 in.

(3) Laying the Tiles. The roof having thus been prepared, the tile-layer is to fasten the tiles with copper nails. The roofer is to see that the tiles are well locked together and that they lie smoothly, and no attempt is to be made to stretch the courses. The tiles are to be laid so that the vertical lines are parallel with each other and at right-angles to the eaves. The tiles that verge along the hips are to be cut close against the hip-boards, and a water-tight joint made by cementing cut hip-tiles to the hip-boards with elastic cement. Each piece of hip-roll is then to be nailed to the hip-board, and the hip-rolls are to be cemented where they lap each other. The interior spaces of hip-rolls and ridge-rolls are not to be filled with the pointing-material.

Cost of Roofing-Tiles.* The prices of tiles vary from $7 to $30 per square, according to the character of the surface-finish and to the pattern. The cost of laying, including asphalt-felt, varies from $5 to $10 per square, according to the pattern of tiles used, the number of layers of felt and the character and extent of the roof. If roofing-tiles are laid on book tiles or on cement, 20% must be added to the cost for laying on wooden sheathing. Fluctuating values of copper make the item of copper nails, when these are used, one of importance.

Sheet-Metal Tiles. Roofing-tiles stamped from sheet steel, plain or galvanized, and also from sheet copper, in imitation of clay tiles, are made by several manufacturers and have been extensively used for factories and buildings of secondary importance. The first cost of these tiles, except those made of copper, is much less than that of clay tiles and they do not require as heavy roof-framing. Tin or galvanized-iron tiles, however, must be painted every few years, so that for a long period of years they probably cost as much as clay tiles and more than slate.

* These prices have advanced and the manufacturers' lists must be consulted.

Tin Roofs

The Sheets. Roofing-plates are made of soft steel of various special analyses or wrought iron (more commonly of the former), covered with a mixture of lead and tin, and are designated TERNE-PLATES, in distinction from plates coated only with tin and therefore called BRIGHT TIN. Roofing-plates are coated by two methods. (1) The original method of coating the plates consisted in dipping the black plates by hand into the mixture of tin and lead, and allowing the sheets to absorb all the coating that was possible; and at least one brand of roofing-tin is still made by this process. (2) The other process, by which the majority of roofing-plates are now made, is known as the PATENT-ROLLER-PROCESS, by which the plates are put into a bath of tin and lead, and are passed through rolls. The pressure of these rolls leaves on the iron or steel a thickness of coating which, to a great extent, determines the value of the plates. These rolls can be adjusted to leave a relatively large amount of coating on the plate, an ordinary coating, or a very scant coating. The heavier the coating the more valuable the plate. Some makers employ a variation of this patent process, by which the plates are given an extra dip, by hand, in an open pot, to give a HAND-DIPPED FINISH. It is claimed that hand-dipped plates will last much longer than those made by the new process, although the latter process is much more extensively used and many good roofing-sheets are made by it.

Brands. The best roofing-plates always have the BRAND stamped on them, and as the manufacturers have a pecuniary interest in keeping up the reputation of these brands, the only way of being sure of a good tin roof is to specify a brand of tin that has a reputation for quality and durability. Some of the best-known brands are Taylor's Target-and-Arrow (formerly Old Style); Merchant's Old Method, MF; Follansbee's Banfield Process; and Margaret. Machine-made plates are usually stamped with the weight of coating per box of 112 sheets, 28 by 20-in size.

Sizes of Sheets. The common sizes of tin plates are 10 by 14 in and multiples of that measure. The sizes generally used are 14 by 20 in and 28 by 20 in. The larger size is the more economical to lay, and hence roofers prefer to use it; but for flat roofs the 14 by 20-in size makes the better roof.

Thicknesses of Sheets. Terne-plates are made in two thicknesses, IC, in which the iron body weighs about 50 lb per 100 sq ft, and IX, in which it weighs 62½ lb per 100 sq ft. For roofing, the IC, or lighter weight, is to be preferred, because the seams do not contract and expand as much as they do when the thicker plates are used. For spouts, valleys and gutters, however, IX plates should always be specified, and should preferably be used for flashings, as they are stiffer and less liable to be dented or punched. The thickness of the iron does not add to the durability of the plates, as this depends entirely upon the tin coating.

Weights of Sheets. The standard weight of 14 by 20-in IC terne-plates is 107 lb for 112 sheets, the number usually packed in one box, and of 14 by 20-in IX sheets, 135 lb. The 28 by 20-in sheets should weigh just twice as much. The black sheets, before coating, should weigh, per 112 sheets, from 95 to 100 lb for IC, 14 by 20-in sheets, and from 125 to 130 lb for IX, 14 by 20-in sheets. The difference between the weights of the black sheets and finished sheets is the weight of the tin. A heavily coated tin should weigh from 115 to 120 lb per 112 sheets for IC, 14 by 20-in sheets, and from 145 to 150 lb for IX, 14 by 20-in sheets. The 28 by 20-in sheets should, of course, weigh twice as much.

The Roof. Roofs of less than one-third pitch are made with FLAT SEAMS and should preferably be covered with 14 by 20-in sheets rather than with 28 by 20-in

sheets, because the larger number of seams stiffens the surface and helps to prevent buckles and rattling in stormy weather. For a flat-seam roof, the edges of the sheets are turned ½ in, locked together and well soaked with solder. The sheets are fastened to the sheathing-boards by cleats spaced 8 in apart and locked in the seams. Two 1-in barbed and tinned-wire nails are used in each cleat. No nails should be driven through the sheets. The seams must be made with great care and sufficient time taken to properly SWEAT the solder into the seams. Steep tin roofs should be made with STANDING SEAMS and with 28 by 20-in sheets. The sheets are first single-seamed or double-seamed and usually soldered together, preferably end to end, into long strips that reach from eaves to ridge. The sloping seams are composed of two UPSTANDS, interlocked at the upper edge, and held to the sheathing-boards by cleats. The standing seams are usually not soldered but simply locked together with the cleats folded in about 1 ft apart. Nails should be driven into the cleats only. The use of acid in soldering the seams of a tin roof should be carefully avoided as acid coming in contact with the bare iron on the cut edges and corners, where the sheets are folded and seamed together, causes rusting. No other soldering-flux but good rosin should ever be used.

Durability of Tin Roofs. A tin roof of good material, properly put on, and kept properly painted, will last from forty to fifty years, or longer. All traces of rosin left on the roof should be removed as soon as the tin is laid and soldered, and one coat of paint should be applied promptly; a second coat should follow two weeks after the first. One or more layers of felt or water-proof paper should be placed under the tin, to serve as a cushion, and also to deaden the noise produced by rain striking the tin. The durability of tin roofing, and especially of tin gutters, valleys and flashings, is generally increased by painting the tin on the back before laying. An excellent paint for tin roofs is composed of 10 lb of Venetian red, 1 lb of red lead and 1 gal of pure linseed-oil.

Maintenance of Tin Roofs. The tin roof should be given one coat of paint after it is laid and an additional coat of paint at four-year or five-year intervals should be amply sufficient to keep its upper surface in first-class condition as long as the building stands. With each painting the roof is fully restored to its original condition. Graphite and tar paints should be avoided on tin roofs. Metallic brown, Venetian red, red oxide or red lead, only, should be used as pigments, with pure linseed-oil. Tinned gutters should be swept clear of accumulations of leaves, dirt, etc., and if water has a tendency to lie in the gutters they should be painted yearly.

Number of Sheets Required to a Square. For FLAT-SEAM ROOFING a sheet of tin 14 by 20 in, with ½-in edges, measures, when edged or folded, 13 by 19 in, or 247 sq in; but its covering capacity when joined to other sheets on the roof is only 12½ by 18½ in, or 231.25 sq in. The number of sheets to a square, therefore, equals 14 400 divided by 231.25 or 63, and an area of 1 000 sq ft requires 625 sheets. A box of 112 14 by 20-in sheets will cover, approximately, 180 sq ft. Sheets 28 by 20 in, when edged or folded, have a covering capacity of 490.25 sq in each. To cover 1 000 sq ft (10 squares) requires 294 sheets. For STANDING-SEAM ROOFING the locks require 2¾ in off the width and 1⅛ in off the length of the sheet. A 28 by 20-in sheet, with the seams on the long edges, will cover 463 sq in. To cover 1 000 sq ft requires 312 sheets.

The Cost* of Tin Roofing varies from $8 to $12 per square, according to the grade of the tin, the locality and nature of the work and the scale of wages. Standing-seam roofs cost about 50 cts a square less than flat-seam roofs. The cost, when 14 by 20-in sheets are used, is about 25% more than for 28 by 20-in

* Variations in cost must be ascertained from manufacturers' lists.

sheets, owing to the greater number of seams; hence, more tin, solder, cleats and work are required.

How a Tin Roof Should be Laid *

The Slope of the Roof. If the tin is laid with a flat seam or flat lock, the roof should have an incline of ½ in or more to 1 ft. If laid with a standing seam, there should be an incline of not less than 2 in to 1 ft. Although tin is used on roofs of less pitch than this and on some which are almost flat, a good pitch is desirable to prevent the accumulation of water and dirt in shallow puddles. Gutters, valleys, etc., should have sufficient incline to prevent water from standing in them or backing up far enough to reach standing seams. Tongued and grooved sheathing-boards of well-seasoned dry lumber are recommended. Narrow widths are preferable, and the boards should be free from holes, and of even thickness. A new tin roof should never be laid over old tin, rotten shingles, or tar roofs. Sheathing-paper is not necessary where the boards are laid as specified above. If steam, fumes, or gases are likely to reach the under side of the tin, some good water-proof sheathing-paper should be used. Tarred paper should never be used. No nails should be driven through the sheets.

Flat-Seam Tin Roofing. When the sheets are laid singly, they should be fastened to the sheathing-boards by cleats, using three to each sheet, two on the long side and one on the short side. Two 1-in barbed-wire† nails should be used to each cleat. If the tin is put on in rolls the sheets should be made up into long lengths in the shop, and the cross-seams locked together and well soaked with solder. They should be edged ½ in, and fastened to the roof with cleats spaced 8 in apart, and the cleats locked into the seam and fastened to the roof with two 1-in barbed-wire nails to each cleat.

Standing-Seam Tin Roofing. The sheets should be put together in long lengths in the shop, and the cross-seams locked together and well soaked with solder. They should be applied to the roof the narrow way, and fastened with cleats spaced 1 ft apart. One edge of the course is turned up 1¼ in at a right angle, and the cleats are installed. The adjoining edge of the next course is turned up 1½ in, and these edges are locked, turned over and the seam flattened to a rounded edge.

Valleys and Gutters. These should be lined with IX tin, and formed with flat seams, the sheets being applied the narrow way. It is important to see that good solder, bearing the manufacturer's name, is used, that it is guaranteed one-half tin and one-half lead, new metals, and that nothing but rosin is used as a flux. The solder should be well sweated into all seams and joints.

Painting. All painting should be done by the roofer. The tin should be painted one coat on the under side before it is applied to the roof. The upper surface of the tin roof should be carefully cleaned of all rosin-spots, dirt, etc., and immediately painted. The approved paints are metallic brown, Venetian red, red oxide, and red lead, mixed with pure linseed-oil. No patent drier or turpentine should be used. All coats of paint should be applied with a hand-brush and well rubbed on. A second coat should be applied two weeks after the first and a third coat one year later.

Caution. No unnecessary walking over the tin roof, or use of it for storage of materials, should be allowed at any time. Workmen should wear rubber-soled

* These suggestions are in accordance with the standard working specifications adopted by the National Association of Sheet Metal Contractors.

† The effect of barbing on the holding power of nails is a disputed question.

bes or overshoes when on the roof. Wherever the slope is steep enough the tin ould be laid with standing seams, which allow for expansion and contraction.

Sizes, Weights, Etc., of Roofing-Tin *

Roofing-tin is usually furnished in two sizes, sheets 14 by 20 in and 28 by 20 in, cked 112 sheets to the box. Target-and-Arrow tin is furnished in three thick-ses: IC thickness, approximately No. 30 gauge, U. S. Standard; IX thickness, proximately No. 28 gauge, U. S. Standard; 2X thickness, approximately No. gauge, U. S. Standard, etc. Weight per 100 sq ft laid on the roof, about 65 lb r IC thickness.

Covering Capacity of Roofing-Tin

lat-Seam Tin Roofing. The following table shows the quantity of 14 by 20-in required to cover a given number of square feet with flat-seam tin roofing. A et 14 by 20 in with ½ in edges measures, when edged or folded, 13 by 19, 247 sq in, but its covering capacity when joined to other sheets on the roof nly 12½ by 18½ in, or 231.25 sq in. In the following table each fractional t of a sheet is counted a full sheet.

o. of square feet	100	110	120	130	140	150	160	170	180	190	200
heets required...	63	69	75	81	88	94	100	106	112	119	125
o. of square feet	210	220	230	240	250	260	270	280	290	300	310
heets required...	131	137	144	150	156	162	169	175	181	187	193
o. of square feet	320	330	340	350	360	370	380	390	400	410	420
heets required...	200	206	212	218	224	231	237	243	249	256	262
o. of square feet	430	440	450	460	470	480	490	500	510	520	530
heets required...	268	274	281	287	293	299	305	312	318	324	330
o. of square feet	540	550	560	570	580	590	600	610	620	630	640
heets required...	337	343	349	355	362	368	374	380	386	393	399
o. of square feet	650	660	670	680	690	700	710	720	730	740	750
heets required...	405	411	418	424	430	436	442	448	455	461	467
o. of square feet	760	770	780	790	800	810	820	830	840	850	860
heets required...	474	480	486	492	499	505	511	517	523	530	536
o. of square feet	870	880	890	900	910	920	930	940	950	960	970
heets required...	542	548	554	561	567	573	579	586	592	598	604
o. of square feet	980	990	1000	...	...	...	...	...	...	...	...
heets required...	610	617	625	...	...	...	...	...	...	...	...

A box of 112 sheets 14 by 20 in laid in this way will cover 180 sq ft.

lat-Seam Tin Roofing. The following table shows the number of 28 by 20-in ets required to cover a given number of square feet with flat-seam tin roofing. e flat seams edged ½ in take 1½ in off the length and width of the sheet. The ering capacity of each sheet is, therefore, 26½ by 18½ in, or 490.25 sq in. In following table each fractional part of a sheet is counted a full sheet.

The following tables of sizes, weights, covering capacities and costs are adapted from ul data compiled for the use of sheet-metal workers by the N. & G. Taylor Company, ladelphia, Pa.

No. of square feet.	100	110	120	130	140	150	160	170	180	190
Sheets required...	30	33	36	39	42	45	47	50	53	56
No. of square feet.	210	220	230	240	250	260	270	280	290	300
Sheets required...	62	65	68	71	74	77	80	83	86	89
No. of square feet.	320	330	340	350	360	370	380	390	400	410
Sheets required...	94	97	100	103	106	109	112	115	118	121
No. of square feet.	430	440	450	460	470	480	490	500	510	520
Sheets required...	127	130	133	136	139	141	144	147	150	153
No. of square feet.	540	550	560	570	580	590	600	610	620	630
Sheets required...	159	162	165	168	171	174	177	180	183	186
No. of square feet.	650	660	670	680	690	700	710	720	730	740
Sheets required...	191	194	197	200	203	206	209	212	215	218
No. of square feet.	760	770	780	790	800	810	820	830	840	850
Sheets required...	224	227	230	233	235	238	241	244	247	250
No. of square feet.	870	880	890	900	910	920	930	940	950	960
Sheets required...	256	259	262	265	268	271	274	277	280	282
No. of square feet.	980	990	1000	...	...	...	...	...	...	...
Sheets required...	288	291	294	...	...	...	...	...	...	...

A box of 112 sheets 28 by 20 in laid in this way will cover 381 sq ft.

Standing-Seam Tin Roofing. The following table shows the number of 14 b
20-in sheets required to cover a given number of square feet with standing-se
roofing. The standing seams, edged 1¼ and 1½ in, take 2¾ in off the wid
and the flat cross-seams, edged ⅜ in, take 1⅛ in off the length of the sheet. T
covering capacity of each sheet is, therefore, 11¼ by 18⅞ in, or 212.34 sq in. I
the following table each fractional part of a sheet is counted a full sheet.

No. of square feet.	100	110	120	130	140	150	160	170	180	190
Sheets required...	68	75	82	89	95	102	109	116	123	129
No. of square feet.	210	220	230	240	250	260	270	280	290	300
Sheets required...	143	150	156	163	170	177	184	190	197	204
No. of square feet.	320	330	340	350	360	370	380	390	400	410
Sheets required...	218	224	231	238	245	251	258	265	271	279
No. of square feet.	430	440	450	460	470	480	490	500	510	520
Sheets required...	292	299	306	312	319	326	333	340	346	353
No. of square feet.	540	550	560	570	580	590	600	610	620	630
Sheets required...	367	374	379	387	393	401	407	414	421	428
No. of square feet.	650	660	670	680	690	700	710	720	730	740
Sheets required...	441	447	455	462	468	475	482	489	495	501
No. of square feet.	760	770	780	790	800	810	820	830	840	850
Sheets required...	515	523	529	536	543	550	557	563	570	577
No. of square feet.	870	880	890	900	910	920	930	940	950	960
Sheets required...	590	597	604	611	618	623	630	637	644	651
No. of square feet.	980	990	1000	...	...	...	...	...	...	...
Sheets required...	665	672	679	...	...	...	...	...	...	...

A box of 112 sheets 14 by 20 in laid in this way will cover 165 sq ft.

Standing-Seam Tin Roofing. The following table shows the number of 28 by ·in sheets required to cover a given number of square feet with standing-seam ofing. The standing seams take 2¾ in off the width, and the flat cross-seams, ged ⅜ in, take 1⅛ in off the length of the sheet. The covering capacity of h sheet is, therefore, 26⅞ by 17¼ in, or 463.59 sq in. In the following table h fractional part of a sheet is counted a full sheet.

lo. of square feet.	100	110	120	130	140	150	160	170	180	190	200
heets required...	32	35	38	41	44	47	50	53	56	59	62
lo. of square feet.	210	220	230	240	250	260	270	280	290	300	310
heets required...	65	68	71	74	77	80	84	87	90	94	97
lo. of square feet.	320	330	340	350	360	370	380	390	400	410	420
heets required...	100	103	106	109	112	115	118	121	125	128	131
o. of square feet.	430	440	450	460	470	480	490	500	510	520	530
heets required...	134	137	141	144	147	150	153	156	159	162	165
o. of square feet.	540	550	560	570	580	590	600	610	620	630	640
heets required...	168	171	174	177	180	184	187	190	193	196	199
o. of square feet.	650	660	670	680	690	700	710	720	730	740	750
heets required...	202	205	208	211	214	218	221	224	227	230	233
o. of square feet.	760	770	780	790	800	810	820	830	840	850	860
heets required...	236	239	242	245	249	252	255	258	261	265	268
o. of square feet.	870	880	890	900	910	920	930	940	950	960	970
heets required...	271	274	277	280	283	286	289	292	296	299	302
o. of square feet.	980	990	...	...	...	...	...	...	...	...	...
heets required...	305	308	...	...	...	...	...	...	...	...	...

A box of 112 sheets 28 by 20 in laid in this way will cover 360 sq ft.

aying the Long or Short Way. Sheets 14 by 20 in can be laid either the or short way. The best roof is made by laying the sheets the 14-in way; ilarly, in using the 28 by 20-in sheets, they should always be laid the 20-in , that is, with the short dimension crosswise.

Cost of Roofing-Tin

Cost of Tin for Standing-Seam Roofing

Sheets 28 by 20 in. Price per box and per square foot

hen tin costs per box......	$11.00	$11.50	$12.00	$12.50	$13.00	$13.50	$14.00	$14.50	$15.00	$15.50
anding-seam roofing costs per sq ft.....	0.0297	0.0310	0.0324	0.0337	0.0351	0.0364	0.0378	0.0391	0.0404	0.0418
hen tin costs per box......	16.00	16.50	17.00	17.50	18.00	18.50	19.00	19.50	20.00	20.50
anding-seam roofing costs per sq ft.....	0.0432	0.0446	0.0459	0.0473	0.0486	0.0500	0.0513	0.0526	0.0540	0.0553
ien tin costs per box......	21.00	21.50	22.00	22.50	23.00	23.50	24.00	24.50	25.00	....
anding-seam roofing costs per sq ft.....	0.0567	0.0580	0.0594	0.0607	0.0621	0.0634	0.0648	0.0661	0.0675	....

e above estimates do not include cost of laying. The cost, using 14 by 20-in sheets, amount to about 25% more than the cost, using 28 by 20-in sheets, owing to the er number of seams. More tin, solder, cleats and work are therefore necessary.

Tin in Rolls, or Gutter-Strips

Number of sheets required per linear foot for 20 and 28-in widths

Feet	Widths 20	Widths 28	Feet	Widths 20	Widths 28	Feet	Widths 20	Widths 28	Hundred feet	Widths 20	Widths 28
1	1	1	35	16	23	69	31	44	2	89	[illegible]
2	1	2	36	16	23	70	32	45	3	134	[illegible]
3	2	2	37	17	24	71	32	45	4	178	[illegible]
4	2	3	38	17	24	72	32	46	5	223	[illegible]
5	3	4	39	18	25	73	33	47	6	267	[illegible]
6	3	4	40	18	26	74	33	47	7	312	[illegible]
7	4	5	41	19	27	75	34	48	8	356	[illegible]
8	4	5	42	19	27	76	34	48	9	401	[illegible]
9	4	6	43	20	28	77	35	49	10	445	[illegible]
10	5	7	44	20	28	78	35	50	11	495	[illegible]
11	5	7	45	20	29	79	36	50	12	540	[illegible]
12	6	8	46	21	29	80	36	51	13	585	[illegible]
13	6	9	47	21	30	81	36	52	14	630	[illegible]
14	7	9	48	22	31	82	37	52	15	675	[illegible]
15	7	10	49	22	31	83	37	53	16	720	1 [illegible]
16	8	11	50	23	32	84	38	54	17	765	1 [illegible]
17	8	11	51	23	33	85	38	54	18	810	1 [illegible]
18	8	12	52	24	33	86	39	55	19	855	1 [illegible]
19	9	12	53	24	34	87	39	55	20	900	1 [illegible]
20	9	13	54	24	34	88	40	56	21	945	1 [illegible]
21	10	14	55	25	35	89	40	57	22	990	1 [illegible]
22	10	14	56	25	36	90	40	57	23	1 035	1 [illegible]
23	11	15	57	26	36	91	41	58	24	1 080	1 [illegible]
24	11	16	58	26	37	92	41	59	25	1 135	1 [illegible]
25	12	16	59	27	38	93	42	59	26	1 170	1 [illegible]
26	12	17	60	27	38	94	42	60	27	1 215	1 [illegible]
27	12	18	61	28	39	95	43	61	28	1 260	1 [illegible]
28	13	18	62	28	40	96	43	62	29	1 305	1 [illegible]
29	13	19	63	28	40	97	44	62	30	1 350	1 [illegible]
30	14	19	64	29	41	98	44	63	31	1 395	1 [illegible]
31	14	20	65	29	41	99	44	64	32	1 440	2 [illegible]
32	15	21	66	30	42	100	45	64	33	1 485	2 [illegible]
33	15	21	67	30	43	....	....	....	34	1 520	2 [illegible]
34	16	22	68	31	43	....	....	....	35	1 575	2 [illegible]

Cost of Tin in Rolls or Gutter-Strips

Labor, solder, paint, rosin and other materials not included

A box of 112 sheets in 28-in roll will cover 175 lin ft
A box of 112 sheets in 20-in roll will cover 248 lin ft
A box of 112 sheets in 14-in roll will cover 350 lin ft
A box of 112 sheets in 10-in roll will cover 496 lin ft

Cost per box (28 by 20 in)..........	$10.00	$11.00	$12.00	$13.00	$14.00	$15 [illegible]
Cost per linear foot, 28 in wide.....	0.05714	0.06285	0.06856	0.07426	0.07998	0.[illegible]
Cost per linear foot, 20 in wide.....	0.04032	0.04435	0.04838	0.05241	0.05644	0.[illegible]
Cost per box (28 by 20 in)..........	$16.00	$17.00	$18.00	$19.00	$20.00	....
Cost per linear foot, 28 in wide	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	

Tin in Rolls. For the convenience of roofers and for rush-orders, Target-and-rrow tin is put up in rolls 14, 20 and 28 in wide. Each roll contains 108 sq ft ibout 63 lin ft, 28 by 20-in sheets laid 20 in wide). The tin is painted on one : both sides, as wanted, with an approved metallic brown paint. The seams e carefully soldered by hand, good 100 to 100 solder and rosin being used as flux.

Slag or Gravel Roofing

The Ordinary Gravel Roofing over boards is formed by first covering the rface of the roof with dry felt (paper) and over this laying three, four, or five yers of tarred or asphaltic felt lapping each other like shingles, so that only ›m 6 to 10 in of each layer are exposed. In laying roofs over concrete the dry lt is omitted, a mopping of pitch is placed directly on the concrete and the first yer of the felt embedded in it.

Flashing against walls, chimneys, curbs of skylights, etc., is done by turning e felt up 6 in against the walls. Over this is laid an 8-in strip with half its dth on the roof. The upper edge of the strip and of the several layers of felt is en fastened to the walls by nailing wooden strips or laths over the felt and into e walls. Metal flashings to protect the felt are better than the wooden strips d should be used when possible. At the eaves and on all exposed edges, etal gravel-stops should be used.

A Better Method of Slag or Gravel Roofing is to lay two plies of tarred felt, pping each other 17 in, and then spreading a coat of pitch over the entire roof. i this again three more layers of felt are laid and then coated with pitch, into iich the crushed slag or screened gravel is embedded.

Specifications for Pitch-Slag or Gravel Roofing. The following specifi-tion-notes * describe the latter method more in detail and also the materials at should be used to secure a first-class job. These roofs are most efficient and irable on comparatively flat inclines. The usual built-up roof consists of suc-ssive layers of saturated felt cemented together and surfaced with coal-tar tch or asphalt, into which is embedded the gravel or slag. Tile is also used as a rfacing material. The saturants used in the felt are generally coal-tar or phalt-compounds.

(1) Specification for Pitch-Slag or Pitch-Gravel Roofing Over Wooden Sheathing

This specification should not be used when the roof-incline exceeds 3 in to 1 ft.

Lay one thickness of sheathing-paper or unsaturated felt weighing not less an 5 lb per 100 sq ft, lapping the sheets at least 1 in.

Over the entire surface lay two plies † of tarred felt, lapping each sheet 17 in er the preceding one, and nail as often as is necessary to hold them in pla til the remaining felt is laid.

Coat the entire surface uniformly with pitch.

* Condensed and adapted from the roofing specifications published by the Barrett mpany and known, in their full form, as "The Barrett Specifications." They can be tained from the manufacturers.

† In the Western States the number of "plies" is construed to mean the total num-r of layers, including dry as well as saturated felt, and the terms 3-ply, 5-ply, etc., are reinafter used on that basis. In the Eastern States, 3-ply, 5-ply, etc., usually refers the number of layers of saturated felt. The total number of layers should always be ecified if there is any doubt as to the exact meaning of the term as used in the speci-ations.

Over the entire surface lay three plies of tarred felt, lapping each sheet 22 i over the preceding one and mopping with pitch the full 22 in on each sheet, s that in no place felt touches felt. Do such nailing as is necessary so that all nai are covered by not less than two plies of felt.

Diagram of Gravel or Slag Roofing on Wooden Sheathing

Diagram of Gravel or Slag Roofing on Concrete Base

Spread over the entire surface a uniform coating of pitch, into which, wh hot, embed not less than 400 lb of gravel or 300 lb of slag to each 100 sq ft. Th grains of the gravel or slag are to be from ¼ to ⅝ in in size, and dry and free fro dirt.

The roof may be inspected before the gravel or slag is applied, by cutting a sl not less than 3 ft long at right-angles to the direction in which the felt is lai All felt and pitch is to bear the manufacturer's label.

(2) Specification for Pitch-Slag or Pitch-Gravel for Roofing over Concret

This specification should not be used when the roof-incline exceeds 3 in to th foot. When the incline exceeds 1 in to 1 ft the concrete must permit of nailing nailing-strips must be provided.

Coat the concrete uniformly with hot pitch

Over the entire surface lay two plies of tarred felt, lapping each sheet 17 i over the preceding one, mopping with pitch the full 17 in on each sheet, so th in no place felt touches felt.

Coat the entire surface uniformly with pitch.

Over the entire surface lay three plies of tarred felt, lapping each sheet 22 i over the preceding one and mopping with pitch the full 22 in on each sheet, that in no place felt touches felt.

Spread over the entire surface a uniform coating of pitch, into which wh

hot, embed not less than 400 lb of gravel or 300 lb of slag to each 100 sq ft. The grains of the gravel or slag are to be from ¼ to ⅝ in in size, and dry and free from dirt.

The roof may be inspected before the gravel or slag is applied, by cutting a slit not less than 3 ft long at right-angles to the direction in which the felt is laid. All felt and pitch is to bear the manufacturer's label.

Notes on Slag and Gravel Roofing. The difference between slag and gravel roofing is that for the former crushed slag is used instead of gravel. The greater the number of plies of tarred felt, the greater the amount of pitch that it is practical to use, and it is the pitch that gives life to the roof. As there are several different weights and qualities of tarred felt, a specification should state either the minimum weight per 100 sq ft, single thickness (the most practical weight is from 14 to 16 lb), or some known quality, such as Barrett's "Specification Tarred Felt." Felt weighing less than 12 lb per 100 sq ft is not economical even on the cheaper work. To comply with the Barrett specification the materials necessary for each 100 sq ft of completed roof are approximately as follows:

Over boards	Material	Over concrete
108 sq ft	Sheathing-paper	None
80 to 85 lb	Specification tarred felt	80 to 85 lb
120 to 160 lb	Specification-pitch	180 to 225 lb
400 lb	Gravel	400 lb
300 lb	Slag	300 lb

In estimating felt, the average weight is practically 15 lb per 100 sq ft, single thickness, and about 10% additional is required for laps. In estimating pitch the weather-conditions and expertness of the workmen will affect the amount necessary for the moppings and for a proper embedding of the gravel or slag. As there are several qualities of pitch, a specification should either specify it by name, such as "Specification-Pitch" or "Straight-Run Coal-Tar Pitch," or in specifying asphalt-pitch, the brand or origin should be plainly defined. The use of an under-layer of sheathing-paper next to board-sheathing is mainly for the purpose of preventing any pitch which might penetrate the felt from cementing the roofing to the sheathing. It is also of value in preventing the drying out of the roof from below through open joints. Where a less expensive roof is desired, four plies or three plies of saturated felt may be used. With the four plies there should be used from 90 to 100 lb of pitch per 100 sq ft of completed roof; and with the three plies from 70 to 80 lb of pitch.

Durability of Slag or Gravel Roofs. These roofs, mentioned in the preceding paragraph, will last from five to ten years, or even longer, depending upon the quality of the materials used and the care with which they have been applied. Roofing put on strictly as provided for in the standard specifications will last twenty years or more, and if a tile surface is used, instead of gravel or slag, the roofing will last as long as the structure itself.

Resistance to Fire, Acid-Fumes, Etc. The fire-resisting properties of the slag or gravel roof are due principally to the incombustible material on the surface. It is claimed that the gravel or slag tends to prevent the successive layers of felt and pitch from burning and the whole mass has a blanketing influence on fires originating within the building. Some carefully conducted tests seem to indicate that gravel roofing protects a wooden roof better than tin. The general effect of a fire upon gravel roofing is to soften the pitch or asphalt in the roofing,

to burn out the inflammable oil in them and to cause the residue to swell and in a porous, incombustible coke. This type of roofing is not attacked by corrosive gases or acid-fumes, and is used extensively on railroad-roundhouses and other structures where the conditions are particularly severe. Coal-tar or tar should not be added to the pitch to soften it.

Guarantee. Roofers generally give a five-year guarantee with gravel roof

Cost* of Pitch-Slag or Gravel Roofing. The cost of this type of roofing varies greatly, depending on the location, size and quality of the work, the extremes being approximately \$2.50 and \$3.50 per square for three-ply, \$3.50 and \$4.50 per square for four-ply, and \$4.50 and \$7.00 per square for five-ply roofs.

Asphalt-Gravel Roofing

Asphalt-Gravel or Asphalt-Slag Roofing differs from coal-tar roofing principally in the substitution of asphalt or asphaltic cement for the coal-tar pitch, for saturating the felt as well as for mopping and surface-coating. It is claimed that the oils of asphalt do not evaporate as quickly as do those of coal-tar pitch under ordinary temperatures and that therefore the flexibility and life of asphaltic felt and coatings are not as quickly destroyed. As a matter of fact, asphalt roofs do not always last longer than some coal-tar roofs, but the chances are that they will last fully as long and possibly longer, depending upon the quality of the materials and the workmanship. The asphalt used for roofing is obtained principally from the island of Trinidad.

Specifications for Asphalt Roofing.† The following specifications were prepared by the above-named company and are for Warren's heavy standard Anchor-brand roofing. The manner of laying the felting differs from that ordinarily employed for coal-tar roofing.

(1) Specification for Asphalt-Gravel Roofing Over Wooden Sheathing

Cover the roof with two thicknesses of Warren's Composite roofing-felt manila-paper side down, lapping each sheet 17 in over the preceding one, and securing with nails through tin discs about 2½ ft apart.

Over the entire surface of the Composite felt thus laid, mop an even coating of Warren's Anchor Brand roofing-cement, into which, while hot, lay two thicknesses of Anchor Brand felt, lapping each sheet 17 in over the sheet preceding, sticking these laps the full width with hot Anchor cement and securing with nails through tin discs not more than 20 in apart.

Over the entire surface of the felt thus prepared, spread an even coating of the cement, covering it immediately with a sufficient body of well-screened, dry gravel or crushed slag.

If the roofing is applied in cold weather the gravel or slag must be heated.

Slag only should be used if the incline of the roof exceeds 3 in to the foot.

All layers of felt must be turned up at least 4 in over battlement-walls, skylight-curbs, or any projections raised above the roof.

(2) Specification for Asphalt-Gravel Roofing Over Concrete

The concrete foundation is to be smooth and perfectly graded to carry the water to the outlets or gutters.

Over the entire surface of the concrete first mop a smooth, even coating of Eclipse Asphalt cement, into which, while hot, lay two thicknesses of Warren's Anchor Brand roofing-felt, lapping each sheet 17 in over the sheet preceding

* These prices have advanced and the manufacturers' lists must be consulted.

†The asphalt-roofing materials manufactured by the Warren Chemical & Manufacturing Company of New York have been used for many years and have given good satisfaction.

Mop back for the full width between the laps of the felt thus laid, with War-n's Anchor Brand roofing-cement.

Over the entire exposed surface of the felt mop an even coating of said Anchor ment, into which, while hot, lay two thicknesses of Anchor Brand felt, lapping ch sheet 17 in over the sheet preceding, and sticking these laps thoroughly the ll width with hot cement.

Over the entire surface of the felt thus prepared, spread an even coating of the ment, covering it immediately with a sufficient body of well-screened, dry avel or crushed slag.

If the roofing is applied in cold weather, the gravel or slag must be heated. Slag only should be used if incline of roof exceeds 3 in to the foot. On steep rfaces nailing-strips should be provided in the concrete, unless the latter is fficiently soft to admit of nailing. All layers of felt must be turned up at ast 4 in over battlement-walls and skylight-curbs, or any projections raised ove the roof.

Cost of Asphalt-Gravel or Slag Roofing. Asphalt-gravel roofing costs a tle more than pitch-gravel roofing of the same grade. (See Cost of Pitch-ag or Gravel Roofing, page 1598.)

Roof-Incline.* Asphalt-gravel or asphalt-slag roofing should not be applied roofs which are steep enough to make the material run in hot weather. The anufacturers of various roofings will guarantee the permanency of their roofings r certain maximum slopes.

Prepared Roofing. There is a large number of so-called PREPARED ROOFINGS READY ROOFINGS, which are made by cementing together two, three, or more yers of saturated felt or felt and burlap and then coating the combination either ith a hard solution of the same cementing material, or with hot pitch or asphalt to which is embedded sand or fine gravel. These roofings are commonly put in rolls 36 in wide and are applied by lapping the strips 2 in with a coat of menting material between, and nailing every 2 or 3 in with tin-capped roofing-ils. A sufficient quantity of cement, nails and tin caps is packed in the middle the rolls. The particular advantage of these roofings is that no previous expe-ence is required for laying them and no kettles are required; for this reason they e extensively used in the country, and on railroad-shops, factories, and mill-ildings. In cities there is no particular advantage in using them except for ofs that are too steep for coal-tar pitch, as they cost on the roof about the same good gravel roofing. Many of these prepared roofings are as durable under or-nary conditions as the light-weight gravel roofs. In Colorado, however, it has en found that they are badly damaged by severe hail-storms, probably owing the lack of the protecting gravel. For roofs having a rise of 1 in or more to e foot, these roofings make economical and durable roofs, and for some build-gs are to be preferred to other materials.

Corrugated Iron and Steel Sheets

Corrugated Sheets of iron and steel are very extensively used for the roofing d siding of mills, sheds, grain-elevators and warehouses. The best grades of rugated sheets are now made of double-refined box-annealed iron or steel.†

* The Editor has been notified by the Warren Chemical & Manufacturing Company, w York, that when put on according to their directions, their Anchor Brand roofing has n successfully used on relatively steep surfaces where the slope was as high as 9 in to the t.

† It is claimed that "the life of a genuine PUDDLED-IRON sheet when exposed only to pure air and natural elements is from five to eight times longer, and when exposed to

The corrugations are usually made lengthwise of the sheet, either by passing them through rolls or by pressing the plain sheets in a press made to give the desired corrugations. It is claimed that the latter method gives the more perfect and uniform corrugations. The weight and thickness of the metal is represented by the gauge-number of the black sheets from which the corrugated sheets are made. The standard gauge* for sheet iron and steel in this country is that established by act of Congress, March 3, 1893. (See page 402.)

Gauges. The following table gives the weights and thicknesses of the different gauges, from No. 7 to No. 30, for flat BLACK SHEETS. The gauge extends from No. 7-0, ½ in thick, up to No. 40, 0.005469 in thick, but sheet steel is not commonly made thinner than No. 30, and above 3/16 in, the thickness is generally designated by fractions of an inch. Section 3 of the act of Congress provides that in the practical use and application of this gauge, a variation of 2½% either way may be allowed.

United States Standard Gauge for Sheet Iron and Steel *

Number of gauge	Thicknesses: Approximate thickness in fractions of an inch	Thicknesses: Approximate thickness in decimal parts of an inch	Weights: Weight per square foot in ounces, avoirdupois	Weights: Weight per square foot in pounds avoirdupois
7	3/16	0.1875	120	7.5
8	11/64	0.171875	110	6.875
9	5/32	0.15625	100	6.25
10	9/64	0.140625	90	5.625
11	1/8	0.125	80	5.0
12	7/64	0.109375	70	4.375
13	3/32	0.09375	60	3.75
14	5/64	0.078125	50	3.125
15	9/128	0.0703125	45	2.8125
16	1/16	0.0625	40	2.5
17	9/160	0.05625	36	2.25
18	1/20	0.05	32	2.0
19	7/160	0.04375	28	1.75
20	3/80	0.0375	24	1.50
21	11/320	0.034375	22	1.375
22	1/32	0.03125	20	1.25
23	9/320	0.028125	18	1.125
24	1/40	0.025	16	1.0
25	7/320	0.021875	14	0.875
26	3/160	0.01875	12	0.75
27	11/640	0.0171875	11	0.6875
28	1/64	0.015625	10	0.625
29	9/640	0.0140625	9	0.5625
30	1/80	0.0125	8	0.5

Galvanizing the Sheets adds approximately 2½ oz per sq ft to the above weights. The regular sizes of the corrugations are 2½, 1¼, ¾ and 5/16 in, measured from center to center. Besides these sizes, 5-in, 3-in and 2-in corru

sulphurous and other gases from ten to twenty times longer. than a sheet of steel or semi-steel of the same gauge, or a light-gauge sheet made from pure puddled pig-iron; and that it will wear longer than steel sheets of the heaviest gauges, or galvanized sheets of the same gauge."

* For other gauges, see pages 401, 402, 403, 1469, 1473, 1509, 1510 and 1512.

corrugated roof connectors

› corrugating companies. Corrugated sheets are ft, 7-ft, 8-ft, 9-ft and 10-ft lengths. Sheets can ι cost of 5% extra. The 8-ft length, however, is lth of the sheets, as a rule, is 24 in between cen- so that the covering width is 24 in when one ap. This applies to all sizes of corrugations, all ider sheets. The 2-in, 2½-in and 3-in corrugated m No. 16 to No. 28, the 1¼-in corrugated sheets corrugated sheets from No. 24 to No. 28 and the 26, 27 and 28 only. No. 28 gauge is the one com- ıe sheets are generally painted with a red mineral ınized sheets, also, can be obtained if desired. y the square (100 sq ft), measuring the actual gated sheets.

ated-Steel Roofing *

roofs, either 3-in, 2½-in, or 2-in corrugations :he most common size. The thickness or gauge ›en the supports on which the sheets are laid. be laid on close sheathing, or strips not more The maximum distances between supports for ı: †

½ ft, center to center.

n 2 to 3 ft, center to center.

ft, center to center.

For No. 16 gauge, 5 to 6 ft, center to center.

The least pitch which should be given to roofs that are to be covered with orrugated sheets is 3 in to the foot, and for trussed roofs it is not desirable to

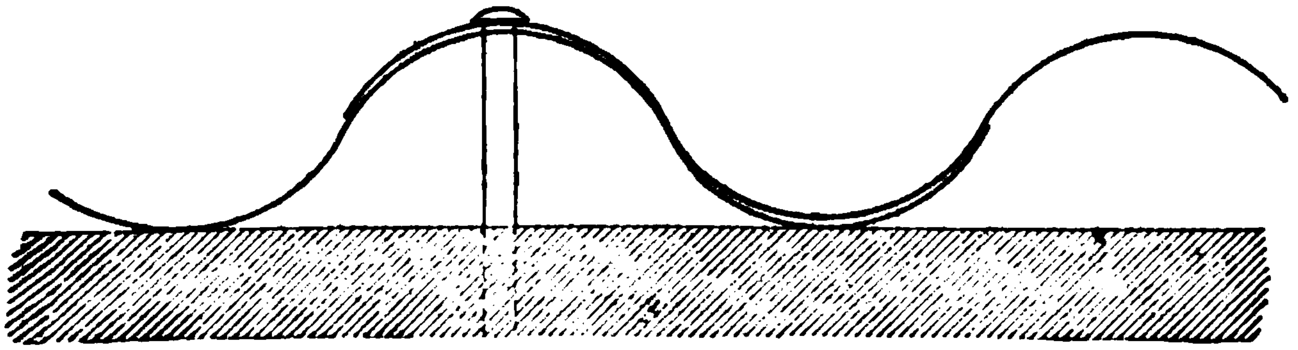


Fig. 1. Approved Method of Laying for Side Lap

ave less than a one-fourth pitch (6 in to the foot). When laid on a roof, corru- ated sheets should have a lap at the lower end of from 3 to 6 in, according to he pitch of the roof. For a ⅓ pitch, a 3-in lap is used; for a ¼ pitch, a 4-in ıp; and for a ⅙ pitch, a 5-in lap. For the side lap it is recommended that ach alternate sheet be laid upside down and lapped as shown in Fig. 1. By his method, when water is blown through the first lap, it will stop and not pass he half lap, but run down and out at the end of the sheet. A great deal of oofing, however, is laid as in Fig. 2. In applying to sheathing or wooden strips, he sheets are secured by nailing through the tops of the corrugations, the nails eing driven through every alternate corrugation at the ends, and about 8 in

* Much practical information regarding the use of corrugated sheets on mill-build- ıgs, with many details, is contained in Steel Mill Buildings and in the Structural En- ineers' Handbook, by Milo S. Ketchum.

† For the strength of corrugated sheets, see the books above mentioned.

apart at the sides. When applied to iron or steel purlins, the side laps shou extend over at least 1½ corrugations, and the sheets should be riveted togeth every 8 in on the sides and at every alternate corrugation at the ends. Th Cincinnati Corrugating Company makes a patent edge-corrugation whic makes a tight joint with a lap of only one corrugation. To fasten the sheets t

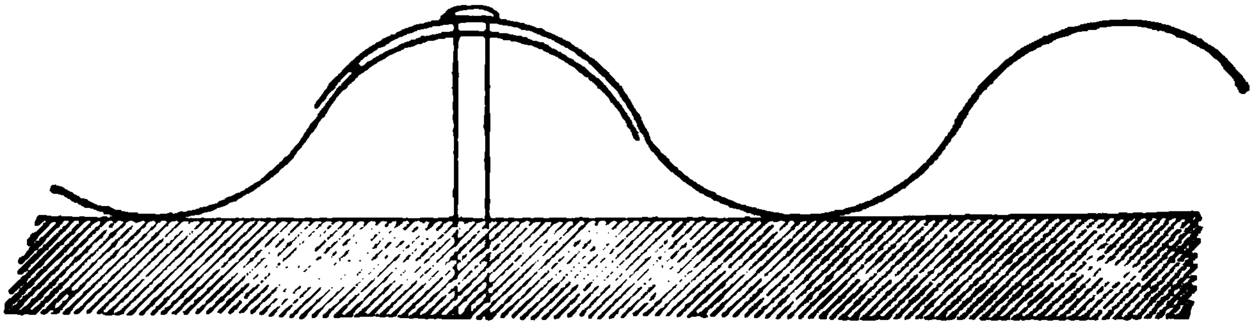


Fig. 2. Common Method of Laying for Side Lap

the purlins, which are usually steel angles, cleats of band-iron, ¾ or ⅞ in wid may be passed around or under the purlins and riveted at both ends to th sheets, as shown in Fig. 3. By contracting or pressing these cleats toward th web, a tight, secure fastening results, which allows for contraction and expansi of the sheets. Cleats, however, are generally used only with channel or Z-ba

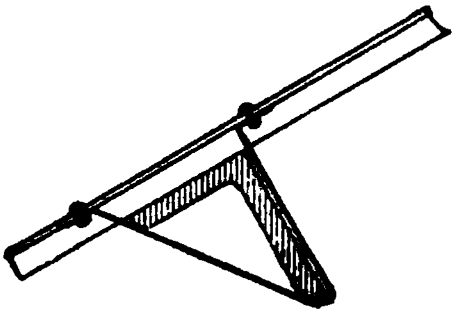


Fig. 3. Sheets Fastened to Angle-purlin by Band-iron Cleats

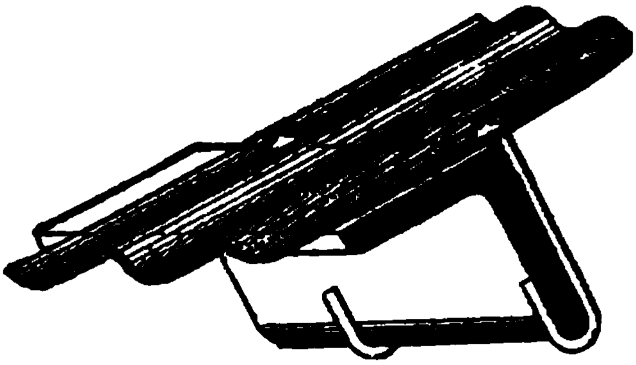


Fig. 4. Sheets Fastened to Angle-purlin by Clinch-nails

purlins. For angle-iron purlins, clinch-nails, made of soft-iron wire, are com monly used, as shown in Fig. 4; they make very satisfactory fastenings.

The following table shows the sizes of clinch-nails to be used with differe sizes of angle-purlins and also the number of nails to the pound in each instanc

Purlin-angles....................	2×2 in	2½×3 in	3½×3½ in	4×4½ i
Lengths of nails.................	4 in	5 in	6 in	7 in
Number of nails per pound......	48	38	33	27

The nails should be placed through the TOP of every second or third corru tion. At the eaves of the building and along the edges of the ventilators spec pains should be taken in fastening the roofing, as these are the places where th force of the wind is the greatest and where it tends to strip the roofing from th purlins. For these parts of the roof the best method of fastening is that show in Fig. 5. These fastenings consist of strips of sheet iron about 2 in wider tha the purlins, made of No. 12 iron and riveted to the purlins with ¼-in rive spaced 10 in apart. To these strips the corrugated sheets are riveted, eve 5 in or every two corrugates, with 6-lb rivets. The method of fastening show in Fig. 6, also, answers very well and is less expensive.

n ordering corrugated sheets an allowance must be made for the laps. The owing table gives the number of square feet necessary to cover one square of

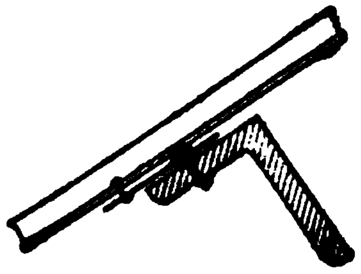


Fig. 5. Approved Fastening for Sheets at Eaves

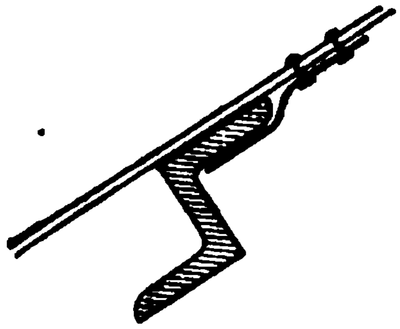


Fig. 6. Alternate Method of Fastening at Eaves

ual surface, using sheets 8 ft long. If shorter sheets are used, the allowance st be slightly increased.

Number of Square Feet of Corrugated Sheets to Cover 100 Square Feet of Roof

nd-laps	1 in	2 in	3 in	4 in	5 in	6 in
	sq ft	sq ft	sq ft	sq ft	sq ft	sq ft
de lap, 1 corrugation	110	111	112	113	114	115
de lap, 1½ corrugations	116	117	118	119	120	121
de lap, 2 corrugations	123	124	125	126	127	128

pproximate Weights in Pounds of 100 Square Feet of 2½-in Corrugated Sheets

auge	No. 28	No. 27	No. 26	No. 24	No. 22	No. 20	No. 18	No. 16
inted	69	77	84	111	138	165	220	275
alvanized	86	93	99	127	154	182	236	291

nti-Condensation Lining. Wherever corrugated steel is laid on purlins no sheathing or paper underneath, if the building is heated, moisture will riably collect on the under side, and if the air in the building is warm and id, considerable dripping will result. To prevent this dripping, it is neces- to protect the under side of the corrugated steel with paper or felt. This be done by first stretching poultry-netting over the purlins, from eaves to e, and wiring the strips together at the edges. Over this should be laid one kness of asbestos paper and one or two layers of saturated felt. The cor- ted steel may then be fastened to the purlins in the usual way. The side may be secured by stove-bolts, with 1 by ⅛ by 4-in plate washers on the r side, to support the lining.

Corrugated Siding

r Siding, either the 2½, 2, or 1¼-in corrugations are used. The 1¼-in size, ever, makes the best appearance. For the laps, 1 in at the bottom and one gation at the sides are sufficient.

r Sheds, etc., the sheets may be nailed to cross-pieces cut in between the s horizontally and spaced from 2 to 3 ft apart, the studs being from 3 to 4 ft

on centers. For elevators, either cross-corrugated sheets or sheets not l than 32 in long should be used. The nails should be driven in the troug each alternate corrugation, 2 in above the lower end of the sheet, which wi 1 in ABOVE the top end of the under sheet. This allows the sheet to slide in 32 in as the building settles, before the nail will strike the upper end of lower sheet. The side lap should not be nailed.

Ceilings. For the ceilings of stores, stables, etc., 3/16 or 5/8-in corrug sheets are much used; and the construction is an excellent one for purpose.

Galvanized Iron. This term is commonly applied to all galvanized s metal. Formerly most of the galvanized sheets had a steel base, but s about 1906 a nearly pure iron, called Toncan Metal, has been largely used sheets of very fine quality. Galvanized sheets come in lengths of 6, 7 and in United States Gauge-Nos. 14, 16, 18, 20, 22, 24, 26, 27, 28 and 30, an widths of 24, 26, 28, 30 and 36 in for all gauges except No. 30, which is made in widths of 24, 26 and 28 in. Sheets of No. 28 gauge are also made in widt 32 and 34 in. The widths commonly carried in stock are 24, 28 and 3 Most of the galvanized iron used for cornices and ornamental work is No gauge. No. 28 is sometimes used for gutters and conductors.

Copper for Roofs

Method of Applying. This is usually in 2½ by 5-ft sheets, making sq ft and weighing from 10 to 14 lb per sheet. It is laid on boards to whi is fastened by copper cleats. No solder is employed, as it is in tin roofs, in horizontal joints, and the horizontal and sloping joints are made by simply o lapping and bending the sheets. The horizontal joints are locked together then tightly flattened down.

MEMORANDA ON TILING

Floor-Tiling and Wall-Tiling

Tile Floors are extensively used in the better class of buildings, and ticularly in those portions which are used by the public, on account of their durability, sanitary qualities and decorative effects. As a matter of fact a tile floor is also cheaper in the long run than a wooden floor if it is subje much wear. The materials used for tiling floors are tiles made from diff grades of clay, marble, slate, glass and rubber. Of these probably the m durable and sanitary are the vitreous clay tiles. For walls and wainscoti glazed tiles, marbles and glass are extensively used.

Floor-Tiles. The following include some of the principle kinds of clay

(1) **Common Encaustic Tiles.** These belong to the cheapest grades, and made of naturally colored clays, red, buff, gray, chocolate and black. T tiles are of a porous, absorbent nature and are used for common floors w sanitary requirements are not exacting.

(2) **Semivitreous Tiles.** These belong to a somewhat better grade than first mentioned and are less porous and absorbent.

(3) **Vitreous Tiles.** These are the hardest tiles known, cannot be scrat by steel or sand, and are non-absorbent and thoroughly aseptic. They are principally for floors requiring a perfect sanitary condition and are manufact in white, blue, gray, green and pink colors of great delicacy.

(?) Ceramic Tiles or Ceramic Roman Mosaic. This material is made of [vit]REOUS clay in tesseral pieces representing the tesseræ of the Roman mosaics. [It i]s made into regular tiles ranging from ½ to ¾-in squares and also in hexagonal [sha]pes from ¾ in to 1 in in size. A rounded LOZENGE TILE is also manu[fac]tured to be laid in tesseral paving. (See, also, Flooring of Mosaic, Terrazzo, [etc.], page 1607.)

[T]he material itself is of great hardness and well suited for work of a monu[men]tal or public character. The even and regular texture of the tesseræ admits [the] adoption of DAMASK DESIGNS which have become identified and associated [with] this material. The minuteness of the tesseræ admits of a great range in [desi]gning and the following of the architectural lines. The ceramic Roman [mos]aic is much preferred to mosaic consisting of natural marbles, because of the [grea]t variety in colors and its greater durability. The vitreous-clay tiles are [imp]ervious to attacks of any acids contained in the atmosphere, while marbles, [espe]cially, are subject to rapid disintegration caused by the sulphuric acid con[tain]ed in the smoke-laden atmosphere of our cities.

[(?]) **Florentine Mosaics and Flint Tiles.** These are the largest and heaviest [tiles] manufactured in this country. They are either plain or inlaid and are in [use] especially in ecclesiastic work on account of their relation to mediæval [decorat]ication. The material is vitreous, annealed and tougher than it is brittle. [It is] also in use for exterior polychrome work.

[(?]) **Aseptic Tiles.** These are large, heavy and thoroughly vitreous tiles used [for i]nstitute work. They are the only vitreous tiles of large size made in this [cou]ntry. As the tiles are large and generally of hexagonal shape, the joint[lin]es are reduced to a minimum, and they are, therefore, especially adapted [for h]ospitals, operating-rooms and wards for contagious diseases.

[E]nameled Tiles, Wall-Tiles and Mantel-Tiles. The following include [som]e of the enameled tiles:

[(?]) **White, Wall-Tiles.** These are glazed tiles for wainscots. They have [a wh]ite, soft body and a surface covered with a clear glaze. The brilliancy of [the] glaze and its reflecting properties make the white wall-tiles especially de[sirab]le for dark passages.

[(?)] **Colored, Glazed or Enameled Tiles.** These tiles are about the same as [the f]ormer in quality; the GLAZE or ENAMEL, however, is stained with metallic [oxid]es, which produces a brilliant decorative effect.

[(?)] **Dull-Satin, etc., Finished, Enameled Tiles.** These are glazed tiles with [DU]LL or BLIND enamel-finish. The dull finish is produced either by sand[blast]ing or by devitrifying enamels. It is principally used for quaint decorative [effect]s in mantel-work.

[(?)] **Glazed Roman Mosaics.** This is a type of enameled tiling which has [great] decorative possibilities. It has the same tesseral texture as the ceramic [mosaic] tiles and is readily applied to wainscots and mantel-work.

[Se]tting of Tiles. Clay tiles are set in Portland-cement mortar as a rule, [and f]looring of this character should always be provided with a substantial [concr]ete base. Ceramic mosaics are sometimes laid on a flexible base. With [this c]onstruction wooden floors can be provided with tile covering, and owing [to th]e elasticity and lightness of the material, floors in elevators, boats and other [buoy]ant structures can be safely tiled.

[Ma]rble Tiles, from 9 to 12 in square, have been extensively used for flooring, [princi]pally on account of their decorative effect. None of the marbles, however, [are as] hard and consequently as durable as the vitreous and ceramic tiles, and

from all practical standpoints the marbles do not make as good floor-co When used, they should be 1¼ in thick and not over 12 in square, and sh bedded in cement on a concrete base. Marbles should not be used for fl in hospitals, as they yield rapidly to the usual antiseptic floor-washes.

Slate, although non-absorbent and not affected even by dilute mineral a is too cold and dingy to commend itself for floor-tiles, but because it is co iently handled in large slabs it is valuable as a cheap base and as a co wiring and pipe-trenches in the floors. As these often follow a wall, it may in the capacity of a border and as such be extended around the floor- Slate slabs for floors should be about 1¼ in thick.

Marbleithic Tiles or Slabs are made of small pieces or chips of marb irregular shapes, set in a backing of sand and Portland cement. After th ment has set, the top surface is rubbed until it becomes flat and smooth. bleithic resembles mosaic or TERRAZZO, except that it is laid in the form of instead of being put down on the floor in a plastic condition. Much objectio been made to TERRAZZO because of the cracks which commonly occur in it, to the slight settlements which are unavoidable in a new building. (See, Flooring of Mosaic, Terrazzo, etc., page 1607.) With tile floors of any ma the joints allow for any slight movement of the floor-construction wi causing visible cracks. By the process of manufacture, marbleithic is much harder than it is possible to make mosaic floors that are laid in a pl condition, so that they have a much better wearing surface. Floors of material have been in use since 1895 and show little if any wear. Marbl tiles are made of various colored marbles and in different sizes, shapes patterns, so that a great variety of effects may be produced. Sanitary co bases, stair-treads, and wainscotings, also, are made of this material.

Cast-Glass Tiles, while quite resistant to a blow when the polish is broken, will break very easily when the surface is scratched. All glass should, therefore, be very thick and small or protected by metal framing.

Novus Sanitary Glass * is a sanitary structural glass manufactured i thicknesses from ½ in up to 2 in and in slabs of all widths and lengths up to in width and 180 in in length. It is made in various colors and designs a the following finishes: natural-fire finish, hone, semipolished and poli It can be worked and handled the same as marble, it is readily drilled and s to accommodate fixtures, etc., and is very handsome in appearance. It i pervious to discoloration and is non-crazing. These qualities make it espe desirable for floors, wainscoting, tables, shelves, etc., in all places where ab lutely sanitary condition combined with a handsome appearance is required

Interlocking Rubber Tiling

General Description. There is an interlocking rubber tiling,† which cause of its being noiseless, non-slippery, and more comfortable to the feet inelastic substances, has met with great favor for floors in banking-ro counting-rooms, vestibules, elevators, stairs, cafés, libraries, churches, etc. elevators it is one of the most durable and practical floors that can be it is also especially and peculiarly adapted for floors of yachts and steam The interlocking feature unites the tiles into a smooth, unbroken she rubber, unlimited in area. The tiles do not pull apart or come up, and being distinct, almost any color-scheme can be employed, the tiles being in a carefully selected variety of colors. The tiles are laid directly over

* Made by the Penn-American Plate Glass Company, Pittsburgh, Pa.
† Manufactured by the New York Belting and Packing Company, New York.

inal floor, like a carpet, except that they are not fastened. Experience has wn that they are very durable. Each tile is 2⅝ in square and ⅜ in thick; tiles are required to the square foot. Rubber nosing for stairs is made to rlock with the tiles.

Cost * of Different Tiles

pproximate Cost. The following prices are approximately the cost, to the le, at the factory. To these should be added the freight and the dealers' its. The cost of laying the tiles on a cement base, in addition to the cost he tiles, should not exceed 25 cts per sq ft.

Floor-Tiles

Kinds of tiles	Factory price per sq ft
mmon encaustic tiles, unglazed	15 cts
treous tiles, white	22½0 cts
lors, large sizes	from 23 to 26 cts
ramic tiles, or ceramic Roman mosaic	from 20 to 35 cts

Wall-Tiles and Mantel-Tiles

Kinds of tiles	Factory price per sq ft
iite glazed wall-tiles	23 cts
lored glazed or enameled tiles	35 cts
ameled tiles, dull satin-finish	50 cts
rbleithic, from 45 cts, upwards, laid	....
nd-made faience, plain colors	from $0.60 to $1

Flooring of Mosaic, Terrazzo,† etc.

looring of Mosaic Work is largely used. (See, also, Ceramic Tiles, or mic Roman Mosaic, page 1605, and Marbleithic Tiles, page 1606.) It is posed of small pieces of stone, marble, pottery or glass, usually laid in some mental design or pattern. A bed of concrete is first laid and the small s of the material used set in a floating of cement and made from ½ to 1 in . When cubes of varicolored marble are used, pressed into the cement ar, it is called ROMAN MOSAIC. A somewhat cheaper flooring is made by ding marble chips of irregular shape over the surface of the cement, ing them into it with plasterers' floats and rolling them with iron rollers. is called TERRAZZO MOSAIC. The following is from the specifications for new Field Museum, Chicago, Ill., D. H. Burnham, architects: "Filling r terrazzo shall be composed 1 part cement, 2 parts sand and 4 parts brick. re concrete filling commences to set spread a ¾-in wearing surface composed arble chips with only enough neat Portland cement to firmly unite the s. Trowel and roll, and after the mortar has set, rub the terrazzo to a oth, even surface and wash clean." "Terrazzo floors in the East cost from 30 cts per sq ft, contractor's profit included."‡

hese prices have advanced and the manufacturers' lists must be consulted.
ee article on Terrazzo Floors, by C. R. Marsh, in Journal of the Society of Construc- f Federal Buildings, July, 1914.
uoted from The New Estimator, by William Arthur, 1914.

ASPHALTUM

Bitumen, Asphaltum, Asphalt. "Bitumen is the name used to denote group of mineral substances, composed of different hydrocarbons, found wid diffused throughout the world in a variety of forms which grade from thin vo tile liquids to thick semifluids and solids, sometimes in a free or pure state, l more frequently intermixed with or saturating different kinds of inorganic organic matter. To designate the condition under which bitumen is fou different names are employed; thus the liquid varieties are known as NAPH and PETROLEUM, the semifluid or viscous as MALTHA OF MINERAL TAR, and t solid or compact as ASPHALTUM OF ASPHALT." *

Asphaltum is found in extensive beds or lake-like deposits on both contine the most notable of these are the PITCH lakes on the island of Trinidad, and Bermundez, Venezuela. It is also found saturating the limestone and sa stone formations in certain localities. Deposits of very nearly pure asphalt are found in Utah, Mexico, Cuba, and various parts of the United Sta ELATERITE, GILSONITE and WURTZILITE are varieties of very nearly pure asp tum.

Asphaltic Roofing-Materials are manufactured principally from Trinid asphalt. These deposits have also been the main source of supply for the a phaltum used in street-paving in the United States.

Rock-Asphalt. The term ROCK-ASPHALT is commonly used to designate t material obtained from the bituminous limestone deposits at Seyssel and Py mont, in the valley of the Rhône, France, in the Val-de-Travers, canton Neuchâtel, Switzerland, and at Ragusa, on the island of Sicily. It is extensi employed for paving purposes throughout Europe, and is considered to mak much more durable pavement than can be made with asphaltum. Ro asphalt is prepared for shipment in two forms: (1) COMPRESSED ASPHALT BLOC which are used for paving in much the same way as stone blocks, and (2) MAS ASPHALT, which is put up in cakes of varying shape, generally bearing the ma facturer's trade-mark.

Mastic-Asphalt. In the Eastern States MASTIC-ASPHALT is used for floors cellars, stores, breweries, malt-houses, hotel-kitchens, stables, laundries, c servatories, public buildings, carriage-factories, sugar-refineries, mills, rinks, and for any place where a hard, smooth, clean, dry, fire-proof and water-pr odorless and durable covering of a light color is required, either in the basem or upper stories. It can be laid over cement concrete, brick, or wood, in sheet without seams; also over cement concrete for roofs for fire-proof build For dwelling-house cellars, especially on moist or filled land, this materia especially adapted, being water-tight, non-absorbent, free from mold or impervious to sewer-gases, and for sanitary purposes, invaluable. Ma asphalt is also valuable for DAMP-COURSES over foundations, and for cove vaults and arches under ground.

Asphalt Floors and Pavements. For floors of cellars, courtyards, laid on the ground, a base of cement concrete 3 in thick should first be laid over this a layer of asphalt from ¾ in to 1½ in thick, according to the us which it is to be put. For ordinary cellar-floors, the asphalt need not be than ¾ in thick; for yards on which heavy teams are to drive, it should 1½ in thick. In specifying asphalt pavement, both the thickness of the conc and of the asphalt should be given; it should also be remembered that ASPH PAVEMENT does not include the CONCRETE FOUNDATION unless so specified.

* Byrne, Inspectors' Pocket Book.

ying asphalt over planks or boards, a layer of stout, dry, but not tarred, sheathg-paper should first be put down and the asphalt laid on this. Asphalt floors stables should be at least 1 in thick. Architects and owners desiring to emoy ROCK-ASPHALT for any of the above purposes should be careful to secure the nuine VAL-DE-TRAVERS, SEYSSEL, or SICILIAN ROCK-ASPHALT, as there are itations which are of but little value.

The Bituminous Sandstones of California have been extensively used for ving streets in Western cities. They are prepared for use as paving-materials crushing to powder. With this powder a considerable proportion of sand gravel is generally mixed and the mixture heated until it becomes plastic; it then spread over the roadways and compressed by rolling.

MINERAL WOOL

Sources of Mineral Wool. There are at least two kinds of mineral wool de in this country. The more common quality is made by mixing certain ds of stone with the MOLTEN SLAG from blast-furnaces and converting the ole mass into a fibrous state. The best slag for the purpose is that which is e from iron. The appearance of the finished product is much like that of ol, being soft and fibrous, but in no other respect are the materials alike. neral wool made from slag appears in a variety of colors, principally white, t often yellow or gray, and occasionally quite dark. The color, however, is d to be no indication of the quality, as all of the peculiar properties of the terial are present in equal proportions in any of the shades. The other kind mineral wool is known as ROCK-WOOL, and is made from granite rock raised to oo° F. It is claimed that as it is absolutely free from sulphur, it is the only rless wool manufactured. It has been approved by the United States War partment. It has the same general appearance as that made from slag, and white in color.

Nature of Mineral Wool. Both of these materials consist of a mass of very , pliant, but inelastic, vitreous fibers interlacing in every direction and ning an innumerable number of minute air-cells. Its great value in the insuon and protection of buildings lies in the number of air-cells which it contains, consequent non-conduction of heat, and its fire-resisting qualities. In wool de from common slag, 92% of the volume consists of air held in minute cells, le in the best grade the proportion of air reaches as high as 96%. This cond air makes it one of the best, if not the best, of the non-conductors of heat. de from these qualities it is very durable and contains nothing that can decay become musty. Being itself incombustible it greatly retards the burning of oden floors or partitions if their inner spaces are filled with it.

Uses of Mineral Wool. The greatest value of this material is as an insulaof heat, but it is also a valuable non-conductor of sound. It is the general nion, however, that it can be considered only as a MUFFLER of the soundes, for there seems to be no practical way in which it can be used so as to arate entirely the floor and ceiling. It would be crushed by laying floorts upon it. As a muffler or filling between the beams, however, there is bably nothing that is superior. In the end, then, it would seem that the t complete insulation from sound, without separate beams, would be obtained FLOATING the flooring on some material like Cabot's Quilt or on a very thick with the spaces between the floor-cleats filled with mineral wool.

Manner of Applying Mineral Wool. Mineral wool, when used alone as -deadening, may be laid on boards cut in between the joists, or on top of thing-lath when that material is used. The wool should be at least 2 in

thick. Again, mineral wool is particularly desirable for filling the spaces betw the studs of outside walls and partitions and between the rafters of roofs. may be used to great advantage, also, in partitions around bath-rooms or wa closets, and around water-pipes when placed in partitions. In outside w and attic roofs, as a protection from the heat of summer or the cold of wi it is of the greatest value. By lathing the under side of the rafters with she ing-lath, and spreading on top a layer of 2 or 3 in of mineral or rock-wool, t comfort of the room is greatly increased. Flat roofs over inhabited rooms m be covered with rough boards and 1¾-in cleats nailed on top, the spaces fi with wool, and the roof-sheathing then nailed to the cleats. This not o greatly increases the comfort of the rooms, but greatly retards the progres fire from the outside. When insulating against heat, nails driven through t insulating material do no harm. When using mineral wool in floors it should packed in very closely, but not jammed so as to break the fibers, which t naturally very brittle. In partitions it is packed between the studs and la so as to completely fill the spaces, the wool being put in after the lathing l reached a height of 2 or 3 ft. More laths are then put on, the spaces filled, a so on to the top. The wool should not be dropped from any considerable heig as the breaking up of the fibers destroys the insulating qualities of the mater In fact the tendency of mineral wool to settle and consolidate, if improperly too loosely packed, is the only drawback, except cost, to its use for insulat The wool behind the lathing will not prevent the plaster from keying.

Cost of Mineral Wool. Mineral wool is sold by the pound, and in estim ing the quantity of wool required, 1 lb per sq ft of filling, 1 in thick, should allowed for ordinary wool and ¾ lb for selected wool.

ESTIMATING THE COST OF BUILDINGS *

Cost of Buildings per Cubic Foot. The method of CUBIC-FOOT VALUES has been used more than any other in estimating the cost of any proposed building, before the plans and specifications are sufficiently complete for taking off the actual quantities. "Comparison of UNIT COSTS is the only scientific criterion by which to judge the economic merit of a structure, a machine or a method of doing work." † Two buildings in the same city, or district built in the same style and for the same purpose, of the same materials, and on the same scale of wages and prices of materials, should cost the same, or very nearly the same, per cubic foot, although one building may be somewhat larger than the other and of different shape. It therefore follows that if we know the COST PER CUBIC FOOT of different classes of buildings, in different localities, we can approximate quite closely the cost of any proposed building by multiplying its cubic contents in feet by the known cost per cubic foot of a similar building already built in that locality.

Size of Building Proportioned to Cost per Cubic Foot. If the cost of a proposed building must be kept absolutely within a certain sum, the size of the building should be proportioned so that the CUBIC CONTENTS shall not exceed the quotient obtained by dividing the amount appropriated by the AVERAGE COST PER CUBIC FOOT of similar buildings. Even then it may be found, when the bids are opened, that they exceed the appropriation; but the excess is often a relatively small percentage of the total cost and the necessary reductions can be made without altering the main features of the building.

Methods of Computation. In estimating the cost by the METHOD OF CUBIC CONTENTS, it is of course necessary that the contents be computed on the same basis, in both the proposed building and the one already built. The cubic contents are generally computed from the basement or cellar-floor, to the average height of a flat roof, or, if there is a pitched roof, the finished portion of the attic is included, or that part which might be finished, mere air-spaces and open porches not being included. Vaults and areas under sidewalks, etc., are generally included as part of the basement. All measurements are to the outside of the walls and foundations. The estimated cost may or may not include the fees of the architect and other experts.

Other Methods of Estimating the Cost of Buildings. The cost of buildings, such as hospitals, theaters, schools, churches, barracks, large stables, etc., is sometimes estimated by the COST PER BED, SITTING, INMATE, etc. Estimates are also based upon the COST PER SQUARE FOOT of GROUND OCCUPIED or of all the FLOOR-SPACE, in certain types of buildings.

* The editor is indebted to E. S. Hand and others for valuable data relating to this subject. Readers are referred to the Handbook of Cost Data for Contractors and Engineers, by H. P. Gillette, The New Building Estimator, by William Arthur, and The Building Estimator's Reference Book, by F. R. Walker. Values given are pre-war values and may be used for relative costs.

† H. P. Gillette, in the preface to his Handbook of Cost Data for Contractors and Engineers.

Data * on Cubic-Foot Values as a Basis for Preliminary Estimates of Building Costs

Notes on Modifying Conditions. Buildings of a given TYPE, such as offic buildings and school-buildings, when similar in construction and finish and bu under similar market-conditions as to cost of labor and materials, are found t be nearly identical in CUBIC-FOOT COSTS. The buildings of any such type do n differ widely in bulk, and this is always very considerable when compared wit such structures as dwellings and small business buildings. This seems on another way of saying that similar causes produce similar effects, but it goes step farther by indicating that the results here are virtually identical; so near so, that the AVERAGE CUBIC-FOOT COST of a certain kind of building can be reli on to produce an estimate within from 3 to 5% of the actual cost of new work o the same kind and under the same conditions. Other types of large structure such as public buildings, hotels, churches and theaters, are less subject to stand ardization because more variable in equipment and finish. This is true also dwellings, shops and other small structures whose lesser bulk, moreover, rende even less possible a close prediction as to their cost. These uncertainties do no however, warrant the rejection of the CUBIC-FOOT-COST METHOD for prelimina estimating. They do indicate that it is less closely approximate for some type than for others. But the degree of uncertainty on even the most variab types may be minimized and should be reduced to perhaps 10% under a caref system of cost-computation. Such system should cover a considerable numbe of examples, taking account of all factors of material influence upon cost in eac type, and must follow a consistently uniform method of determining cubic-foo values.

The Factors Which Influence Cost include the following.

(1) Prevailing market prices of labor and materials.

(2) Type of construction employed, depth and kind of foundations and exist ence of special features such as towers or domes.

(3) Finish: external facing and ornamentation; internal surfacing and deco ration.

(4) Equipment: (*a*) number and complexity of heating, lighting, ventilati sanitary, elevator and other systems; (*b*) extent to which appara or equipment, such as laboratory-devices, opera-chairs, bank-counte etc., is provided for direct use of occupants of building.

(5) Fees of architect and other experts.

(6) Locality. Costs of structures of a given type will vary with the locali because of differing standards of practice and building laws, availabil of building materials, labor, etc.

(7) Other items, developing in the experience of the architect.

The Method of Determining Cubage may either simply recognize GEOMETRICAL VOLUME of the building or, better, may employ a COEFFICI OF VALUE for any part whose cost varies materially from the average. T latter method may be preferred as allowing a closer calculation of variat from known examples. For instance, an unfinished cellar or other story o small light-court would cost less per cubic foot than the remainder of the bu

* The data on this page and page 1613 on cubic-foot values are quoted, by per sion, from notes relating to this subject, compiled by Professor Warren P. Laird the study of a large number of public and private buildings erected in widely sepa districts of the United States. For these buildings Professor Laird acted as the pro sional adviser for the selection of the architect, and in all cases the estimate of the of the buildings was based strictly upon a total number of cubic feet and a fixed cost per cubic foot.

ıg, while a tower or dome of finished basement containing, also, an expensive ɩechanical plant, would cost more. Foundations sometimes cost so much ıat they require figuring to their full depths as though the finished building ere carried down to that level.

Cubic-Foot Costs. Subject to the foregoing considerations, the following ıta on fire-proof buildings were average pre-war values. These unit prices no nger prevail as labor and materials have in some cases almost doubled in cost nce the war. The values must be increased from 50 to 100%, depending upon ıe kind of building.

Construction: steel and terra-cotta, stone and brick facings, complete equipent and superior grade of interior finish:

Type of building	Cents per cubic foot
Office-buildings	32 to 35
Public buildings	40 to 45
School-buildings	20 to 25

Construction: reinforced concrete; facing, common brick; equipment, type ual in such structures; inside finish, the simplest.

Type of building	Cents per cubic foot *
Factories †	14 to 16
Lofts †	15 to 18

ıble for Estimating Roughly the Approximate Cost of Some Small Buildings

Based on prices for labor and materials in 1920. The cost of first-class fireɔof buildings is greater in the Western and Southern States than in the Eastern ıtes, because of the distance from the great steel and material-centers.

Farm and Country Property

	Cost per cubic foot, cents
vellings, frame; small box house, no cornice	10
vellings, frame; shingle roof, small cornice, no sash weights, plain.	12 to 14
vellings, brick; same class	16 to 18
vellings, frame; shingle roof, good cornice, sash weights, blinds (good house)	16 to 18
vellings, brick; same class (good house)	20 to 22
rns, frame; shingle roof, not painted, plain finish	4 to 6
rns, frame; shingle roof, painted, good foundation	6 to 8
res, frame; shingle roof, painted, plain finish	12 to 16
res, brick; shingle roof, painted, good cornice, well finished	16 to 20
linary frame churches and schoolhouses; country	12 to 16

These pre-war values must be increased from 50% to 100%, depending upon the kind uilding.

If such subcontracts as plumbing, heating, lighting-fixtures, elevators, etc., are not uded, of course these figures were reduced. See Cost of Reinforced-Concrete Build-, page 1618.

Brick churches and schoolhouses; country........................ 18
If the roofs are slate or metal, add ¼ ct per cu ft.

City and Village Property

Dwellings, frame; shingle roof, pine floors and finish, no bathroom or furnace, plain finish (good house)........................ 14
Dwellings, brick; same class........................ 15
Dwellings, frame; shingle roof, hard-wood floor in hall and parlor, bath, furnace, and fair plumbing........................ 18
Dwellings, brick; same class........................ 18
Dwellings, frame; shingle roof, hard-wood in first story, good plumbing, furnace, artistic design, some interior ornamentation, well painted........................ 22
Dwellings, brick; with good plumbing, bath, hot and cold water, pine finish, well painted, no hard-wood finish........................ 24

Examples of Actual Costs of Pre-war Buildings per Cubic Foot

In order to illustrate the subject further, examples of buildings erec before the war are given. The lists were furnished through the courtesy of architects of the buildings. Such lists could be indefinitely extended, but th submitted are deemed sufficient to give some idea of the similarities and va tions of costs based upon cubage. With the exception of reinforced-conc buildings, it is probably true that for twenty or twenty-five years (1890 to 19 the cost of buildings increased, with some variations in the rate of increase, the rate of about 1% per year. Costs have increased since the war from 5 100%, depending upon the kind of building.

Examples of the Actual Cost of Pre-war Buildings per Cubic Foot

These buildings were designed by Boring & Tilton

ame and location of building	Date	Height and type	Approximate cost	Cost per cubic foot, cents
emorial Hall, Tome Institute, Port Deposit, Md.	1900	Three stories and basement, fire-proof	$150 000	16
'est Side Branch Library, Cleveland, Ohio	1908	One story and basement, non-fire-proof	85 000	17
tamford Grammar School, Stamford, Conn.	1908	Two stories and basement, non-fire-proof	50 000	15
t. Agatha's School, 87th St. and West End Ave., New York City	1907	Six stories and basement, fire-proof	275 000	29
merican Seamen's Friend Society, Jane and West Sts., New York City	1909	Five stories and basement, fire-proof	200 000	32
astern District Branch Y.M.C.A., Brooklyn, N.Y.	1906–1909	Six stories and basement, fire-proof	255 000	27
arrytown Hospital, Tarrytown, N. Y.	1910	Two stories and basement, non-fire-proof	65 000	26
lair Hospital, Huntingdon, Pa.	1909	Three stories and basement, fire-proof	90 000	20
lizabeth Library, Elizabeth, N. J.	1912	Three stories and basement, fire-proof	100 000	35
pringfield Library, Springfield, Mass.	1908	Two stories, mezzanine and basement, fire-proof	350 000	35
ioux City Library, Sioux City, Iowa	1912	Two stories and basement, non-fire-proof	75 000	21½
11 Park Avenue, New York City	1912	Twelve stories and basement, fire-proof	350 000	45
'nited States Immigrant Station, Ellis Island, New York Harbor. Main building	1898	Two stories and basement, fire-proof	625 000	17½
lospital building	1898	Three stories and basement, fire-proof	150 000	28
lount St. Mary's College, Plainfield, N. J.	1912	Three stories and basement, fire-proof	250 000	22

These buildings were designed by Palmer, Hornbostle & Jones

Name and location of building	Date	Notes	Cubic contents, cubic feet	Approximate cost *	Cost * per cubic foot, cents
Oakland City Hall, Oakland, Cal.	1914	Based on all contracts except for lighting-fixtures	2 999 442	$1 400 000	[illegible]
Allegheny County Soldiers' Memorial, Pittsburgh, Pa.	1911	....	2 855 892	913 721	32
New York State Education Building, Albany, N. Y.	1912	Original contract completed, Dec. 1912	11 281 691	3 744 521	33¼

These buildings were designed by Robert D. Kohn

Name and location of building	Date	Height, character of construction and finish	Cost * per cubic foot, cents
Hermitage Hotel, New York City.	1907	Size of lot, 50 by 100 ft; height, 150 ft; 15 bedrooms and 11 baths on each floor; basement and sub-basement; power, electric light and refrigerating-plants; complete kitchen-equipment; brick and limestone exterior; cement floors.	[illegible]
Trades School Building, Manassas, Va.	1910	Main wing, 50 by 100 ft, two and one-half stories; shop-wing, 75 by 105 ft, one story, brick; shops, mill-constructed roof, cement floor; brick exterior throughout; heating-plant in extension; common wooden-floor construction, tin roof, class-rooms plastered; cheaply built.	[illegible]
Ethical Culture Meeting House.	1910	Height, 100 ft; basement, assembly-room; main floor, auditorium for 1 200 people; two stories above auditorium, Sunday school and offices; limestone exterior; fire-proof construction; oak finish.	[illegible]

Cost * of Some Notable Buildings in New York City. Some of th prominent buildings in the Borough of Manhattan, City of New York, a cluded in the following table. For all these structures the costs per cubi are given. By reason of its height the Woolworth Building may be consid the most notable of the list. It is not only the highest building in Ne City or the United States, but in the world. The cubic contents total 12 000 000 cu ft. Its foundations are carried to rock, which is about below the street-surface. The approximate weight of its steel frame is tons.

* See notes on cost on page 1614.

Cost-Data * of Some Notable Pre-war Buildings in New York City †

			Ground-areas, sq ft	Total floor-areas, sq ft	Cost	Cubic contents, cu ft	Costs per cubic foot, cents
Altman Building [1]	8-story department-store	152	54 850	495 000	....	....	33
Banker's Trust Co.'s Building [1]	39-story office-building	566	9 721	345 000	....	....	70
Heckscher Building [2]	50 E. 42nd St.	317	10 750	147 172	$700 000	1 793 351	39
Masonic Building [3]	19-story lofts and meeting-rooms	292	23 300	432 000	2 250 000	5 701 000	39
Walker-Lispenard Building [4]	24-story telephone-exchange	365	27 580‡	....	....	....	40
Woolworth Building [5]	55-story office-building	792	15 600	790 000	....	7 250 000	62⅙
United States Rubber Co.'s Building [6]	20-story office-building	273	10 800	276 000	1 628 707	3 090 205	52⁷⁄₁₀
The Madison Avenue Building [7]	20-story office-building	288	14 700	315 000	1 300 000	4 708 000	27⅔
Auerbach Candy-Factory [8]	11-story factory	163	35 100	343 680	915 000	5 200 000	17½
Æolian Building [9]	17-story lofts	248	15 700	244 724	2 000 000	4 225 000	47⅓

* See notes on cost on page 1614.
† The editor is greatly indebted to Mr. E. S. Hand and to the architects mentioned for the date for this table. ‡ Typical floor-area.

These buildings were designed by the following architects: [1] Trowbridge & Livingston; [2] Jardine, Hill & Murdock; [3] H. P. Knowles; [4] McKenzie, Voorhees & Gmelin; [5] Cass Gilbert; [6] Carrère & Hastings; [7] Charles A. Valentine; [8] Robert D. Kohn; [9] Warren & Wetmore.

The Grand Central Station, as a complete terminal, is a very complex stru-ture, but there is a distinct part which contains the passenger-concourse and the waiting-rooms, restaurant and other parts that are considered necessary to care for the traffic. The cubic contents of this part total about 14 000 000 cu ft. Other parts of the building are not considered in the present reference. Some interesting facts as to the main station, only, are:

Cost,* about	$8 000 000
Ground-area above street-level, square feet	266 000
Additional station-facilities under street, square feet	80 000
Floor-area devoted to station-purposes, square feet	1 188 000
Cubic contents, about, cubic feet	32 857 800
Steel used in construction, tons	35 767
Weight of largest girder used, tons	30

Costs* of Pre-war Reinforced-Concrete Buildings.†‡ In judging the cost of a building by CUBICAL CONTENT or by AREAS OF FLOORS the shape of the build-ing in plan should be taken into consideration. A long, narrow building will cost more per cubic or square foot than one more nearly square in plan; and in computing costs by the cubic-foot or square-foot unit prices these conditions as well as the judgment and experience of the architect or engineer who makes the estimates affect the accuracy of the results. The following notes quoted from data furnished by the architects and engineers of the buildings mentioned include useful information relating to costs of some reinforced-concrete buildings of different types, erected in Philadelphia and vicinity (1906–1915).

(1) ' A reinforced-concrete building of the FACTORY-TYPE, erected (1914-15) in the City of Philadelphia. It is a concrete cage, with no brick veneer, four stories in height, no basement, size, 60 by 159 ft, stair-shafts and elevator shafts projecting beyond the building; cubical contents, 603 000 cu ft. The cost, without equipment, was 7½ cts per cu ft. Drainage is included in this price, but no plumbing, heating, lighting or elevators. The total floor-area of the building is 40 140 sq ft and the cost per square foot is $1.14½. This is built according to the building laws of Philadelphia.

(2) "A MILL-CONSTRUCTED BUILDING, about the same size as building (1) recently erected in a manufacturing town forty miles from Philadelphia. It is four stories in height and has a part-basement, a wing 30 by 40 ft, and a one story boiler-room and engine-room. The total cubical contents are 524 [illegible] cu ft, and the cost, 6⅛ cts per cu ft. The total floor-area is 37 900 sq ft, and the cost, $0.85⅛ per sq ft. This is without power, heat, or light. There are a few plumbing-fixtures in this building.

"In comparing the costs of the two buildings, it must be borne in mind that one is located forty miles from Philadelphia, and was not erected under the rigid building laws that are in force there. It is usually possible to erect a building of any type at less expense outside of Philadelphia than in that city and this can probably be said of any city where there are no state building codes

(3) "A MILL-CONSTRUCTED BUILDING, three stories in height, erected in 19[illegible] in Camden, N. J., and having 575 044 cu ft. It cost 7 cents per cu ft. It has 38 912 sq ft of floor-area, at a cost of $1.04 per sq ft. This price is without power, heat, light, or elevators, but includes some plumbing.

(4) "The new municipal REPAIR-SHOP of the City of Philadelphia. This is a reinforced-concrete building with brick veneer of an ornamental type, and cost 9⅛ cts per cu ft for 1 080 591 cu ft or $1.74 per sq ft for 57 323 sq ft of total

* See notes on costs on page 1614. Prices given must be at least doubled (1920).

† Valuable data on this subject have been furnished the Editor by Ballinger & Perrot the architects and engineers of the five reinforced-concrete buildings described.

‡ See, also, page 1613.

r-area. This is without plumbing, power, heat, light, or elevators. The tively high cost per square foot for this building is due to the fact that the ne run-way takes up a considerable portion of the building, so that a floor mitted where the crane is placed, and the floor-area accordingly reduced.

5) "The new building for the AUTOMOBILE CLUB of Philadelphia. This is hree-story building, of reinforced-concrete cage-construction, and contains 1 966 cu ft, at a cost of 10⅝ cts per cu ft. The total floor-area is 90 602 t, costing $1.54 per sq ft. This is without power, heat, light, or any equipnt, but includes plumbing. The shape of this building favors economy of struction, as it is nearly square in plan."

n summing up the conclusions arrived at in regard to the average costs of forced buildings, E. G. Perrot states * that the cost can best be considered classifying them under three general heads:

1) Warehouses and manufactories. Cost, from 8 to 11 cts per cu ft.

2) Stores and loft-buildings. Cost, from 11 to 17 cts per cu ft.

3) Miscellaneous buildings, such as school-houses, hospitals, etc. Cost from to 20 cts per cu ft.

Cost of Mills and Factories Built on the Slow-Burning Principle. data relating to total and unit costs of buildings of this type, see Chapter II, pages 802 to 810.

rcentages of Cost of Items of Construction in Fire-Proof Buildings

The tables † on the following six pages show, on pages 1620 to 1625, the ISION OF THE COSTS of fire-proof buildings among the different materials and ts of the construction, the data having been furnished the compiler by hitects and builders in the cities mentioned in the tables. Each column of ues in the tables gives the data for an individual building, except the values New York City, in the second, third and fifth columns, which show the rages for a large number of buildings. The tables on the first four pages lude only buildings approximating closely the standard specifications of the tional Board of Fire Underwriters. The tables show that the foundations l steel frames, the only parts little damaged in conflagrations, represent, proximately, only 25% of the entire sound value of a building. For example, the tables on the first four pages, the average cost of all the foundations is , while the average cost of the steel frames is 17.88%. The tables show, o, on pages 1624 and 1625 the percentages of cost of the classified items of nstruction of eight buildings damaged by the Baltimore conflagration (1904), averages of these eight buildings being given in the last column.

* See "Comparative Costs of Reinforced Concrete Buildings," by E. G. Perrot, in oceedings of the National Association of Cement Users, Vol. V, 1909. See, also, notes costs on page 1614.

† The tables on the first four pages were compiled by F. J. T. Stewart, Continental surance Company, and those on the last two pages by the Baltimore Committee of e National Board of Fire Underwriters. All are reproduced, by permission, from J. K. eitag's Fire Prevention and Fire Protection. Those parts of the Baltimore tables ich gave the proportion of fire-damage to sound value of the various items have been nitted as this article of the Pocket-Book deals more especially with original costs.

Classified construction	New York Ofcs	Ofcs	Hotel	Ofcs	W.H.	Chicago Ofcs	Ofcs	Ofcs	Ofcs	Merc	Merc	Baltimore Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs
Height, in stories	10			12								11	12	7	14	9	8	10	7
Cost, cents per cubic foot *				40		30 3	34 8	32 3		14	23 6								
Foundations						4 4	2 34	5 67		6.00	10 74	5 62	4 37	7 25					4 3
ing												1 22	1 25	2 07					4 3
[illegible]													1 06						
l concrete												4 4	0 74	5					
e pier-caps													0 65						
[illegible]													0 13	0 18					
Steel frame	24.0	18 4	16 0	22.90	36 5	10.70	14.0	11.86	10 6	27 6	38 66	13 6	14 54	10 52	9.9	9	10.1	11.4	17 5
Material													11 79						
Erection													1.7						
Shop-drawings													0 49						
Painting													0 24						
Teaming													0.32						
Masonwork	26 96	26 2	31 5	20 70	32 5	26 31	27 06	22 3	38 5	27 6	44.79	23 76	28 5	34 15	30 15	31.31	37 2	29 0	35 5
Brick, common												10 1	6 59	11 21				8.98	13 5
Brick, faced or pressed													1 82	1 26					
Brick, enameled													0.86						
ing and pointing													0 21						
[illegible]	1.41			2 06		10.04		5 76				4 34	7 61	5.07	3.9	2 5		3 5	
[illegible]	2.75			1 50			10.30					2 55	4 22	9 34	4.05	7 4	27 3	8.7	9 4
[illegible]				10 90												0 83			
r furring												0 77				1 8			
Floor-arches, roof, etc.						11.06	9 16	5 8			22.08	3 87	4 73	3 68	6 8	6.6	3.4	3.6	6 6
Cinder-concrete filling over arches						1.06	1 30	1.24			0 47	0 57	0 58	0 49					6.
Partitions												1 56	1.88	3 1					
Partitions, cleaning and wrecking																			
Safety-deposit vaults															7 6			5.16	
Miscellaneous scaffolding and wrecking																0 48			
Miscellaneous masonry	93 40					4 13	7 3	9 6			37.44				7.6	11.79	9.5		

* [illegible] upon the kind of building

The figures opposite each item represent percentages of total cost of building

	New York					Chicago						Baltimore							
	20 65	21 2	12.0	20 89	11 25	24 96	24 54	25 04	23 2	20.8	2 26	19 18	24 54	15 96	18 69	26 77	15 23	21 23	12.69
Elevator-plant	5 70			5 28		4 65	7 81	5 48				7 6	7 68	8 15	5	5 68	3 8	7 7	4 02
Plumbing	3 49			4 16		6 58	5 90	6 19			0 85	3 26	3 55	2 61	4 13	3 5	3 7	3 4	3.36
Heating-system	5 85			5 85			0.67	5 88				4 6	7 06	3 54	4.2	3 6	2 2	6.7	4 15
Boiler-plant							1 92	1 86			0 85					1 9			
Lighting-system, wiring and fixtures	5 37			5 6		3 82	2 76					2 54	5 05	1 38	2 03	1 9	1 6	3	1.10
Dynamos, switchboards, etc.								3 34								2 18			
Fixtures							0 32	1 24				0 7	0 49		1 05	0 65			
Mail-chute	0 24					0 24	0 06	0.13				0 48	0 27	0 28	0 18		0 33	0 43	
Filter-plant													0 44						
Refrigerating-plant															0 93				
																1 9			
...e-doors						0 24	0.29	0 32			0.56				0.65		3 6		
Flash-signals and indicators															0 54				
Furniture																5 4			
Ventilation						9 43	4.80	0 60											
Trim and finish	*28 40*	*20 8*	*33.25*	*22.26*	*14.75*	*24 44*	*19 45*	*35 28*	*27.7*	*18.3*	*4 30*	*36 06*	*27 96*	*22 83*	*36 20*	*28 78*	*36 63*	*26.89*	*25 04*
Carpentry, rough						8 90	6 53	8 54			2 86	2 03	2 59	0 22	10 9	9 8	5 4	7 9	1 6
Carpentry, finish	20.50			10			1 88					5 27	4.31	0 51					5 7
Hardware, rough						0.96	1.70	1 5				0 32	0 29	1 7		0.64	0 9	1 16	1 95
Hardware, finish												0 74	0 88	5 1	0 83				
Marble	2 83			9.85		5 02	4 17	7 61			0 25	6 74	7 11	1 42	10 5	5 4	10 1	6 2	2 8
Mosaic	1 25											0 14		1 99			1 2		
Glass						1.02	1 32	3 37				1 44	9	1 13	1 4	7 5	1 4		0 89
Slate																			
Plastering	3.82			2.41		2 97	3 68	2 35				2 94	3 05	2 96	2 9	4 21	3 7	2 7	1 4
Fresco						1 84	3 49	1 78				0 88							
Paint and varnish						1 64	2 05	1 39			0.30	2 00	1 34	1 45	2.9	1 6	1 4	2 9	1.5
Office-partitions (wood and glass)												2 65							
Ornamental iron						1 09	2 58	6 19				9 70	5 87	4.62	6 4	5 0	10 5	5.5	6.8
Skylights and sheet metal						1 00	2 05	2.55			0 98	0 71	1.22	1.54	0.37	0 63	0 83	0 53	2 4
Office-grill												0 5		0 19			1 2		

Classified construction	Boston																	St. Louis	
	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Merc	Merc	Merc	Merc	Merc	Sch'l	Stg	Merc	Merc	Ofcs	Ofcs
Height, in stories	9	11	10	10	11	11	11	11	7	4	6	3	5	3	7	5	6	12	12
Cost, cents per cubic foot *	24	..	35	49	67	41 8	45	34	...	...	21.5	22	29	29 6	..	...	..	...	..
Foundations	10 6	5 13	4.32	7.5	4 42	5 8	...	...	14.2	14	..	11 5	13 5	7 4	...	24 5	...	5	7
...ck-filling	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
......	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
...s and concrete	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
...granite pier-caps	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
...ts	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Steel frame	14.4	17.4	22.5	13	25.7	21.8	21.4	24 6	8.25	8.2	22.1	10.32	14.65	21.2	21.6	14 3	7.95	15.0	15.0
Material	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Erection	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Shop-drawings	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Painting	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Teaming	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Masonwork	41.4	15.0	32 23	41 6	33 60	27 57	34 83	26	33 65	31 45	22 88	30.6	17.42	30.46	68.62	26	38 6	20.0	25.0
Brick, common	27 5	8 3	18 9	15	19 7	17 75	17 7	16 9	29 2	30	16 8	24	9 75	20 1	64	23 8	..	.	..
Brick, face or pressed	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Brick, enameled	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Brick, cleaning and pointing	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Terra-cotta	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Stone	7 1	1 75	8 17	16 6	5.25	3 55	10 4	4 54	4.45	1 45	3 3	6 6	6 95	6 4	4 62	2 14		...	..
Marble	5.8	4 95	5 16	10	8.65	6 27	6 75	4 56	..	...	2 78	.	0 72	3 76	..	0 08	.	.	...
Wall-lining or furring	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Floor-arches, roof, etc.	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Cinder-concrete filling over arches	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Partitions	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Partitions, cleaning and wrecking	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Safety-deposit vaults	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Miscellaneous scaffolding and wrecking	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Miscellaneous masonry	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

[illegible]

The figures opposite each item represent percentages of total cost of building

Classified construction	Boston																	St. Louis	
Equipment	*9.43*	*10.3*	*18.05*	*15.90*	*14.45*	*20.12*	*23.02*	*21.41*	*8.08*	*23.9*	*18.71*	*12.3*	*11.84*	*16.25*	*2.46*	*10.65*	*19.09*	*17*	*16*
Elevator-plant	3.29	12.4	3.5	4.66	3.92	5.56	4.3			1.54	...	5.07	2.44	2.39	...	4.25	6.45	...	...
Plumbing	1.74	4.75	3.4	4	4.8	5.45	2.92			1.85	3.68	2.9	5.07	4.32	...	1.25	1.5	...	...
Heating-system	2.71	8.4	4.83	5.33	2.88	2.48	8.45			9.85	5.5	3.34	2.34	7.64	1.54	1.36		...	...
Boiler-plant	...	...	1.4	...	5.68	2.37	...			10.4	7.45	...	...	...	...	1.07		...	...
Lighting-system, wiring and fixtures	1.74	4.75	3.79	2.00	1.62	4.26	7.34			0.26	2.38	1.03	1.99	1.9	0.92	1.5		...	...
Dynamos, switchboards, etc.	...	...	...	...	...	...	...			...	...	...	...	...	...	...		...	...
Fixtures	...	...	1.13	...	5.55	...	...			...	...	...	...	...	...	1.22		...	...
Mail-chute	...	...	...	...	...	...	...			...	...	...	...	...	...	...		...	...
Filter-plant	...	...	...	...	...	...	...			...	...	...	...	...	...	...		...	...
Refrigerating-plant	...	...	...	...	...	...	...			...	...	...	...	...	...	...		...	...
Safes	...	...	...	...	...	...	...			...	...	...	...	...	...	...		...	...
Vault-doors and safe-doors	...	...	...	...	...	...	...			...	...	...	...	...	...	...		...	...
Flash-signals and indicators	...	...	...	...	...	...	...			...	...	...	...	...	...	...		...	...
Furniture	...	...	...	...	...	...	...			...	...	...	...	...	...	...		...	...
Ventilation	...	...	...	...	...	...	...			...	...	...	...	...		...		...	...
Trim and finish	*20.87*	*22.11*	*22.4*	*19.34*	*17.16*	*24.73*	*21.3*			*28.27*	*36.32*	*26.36*	*41.35*	*24.92*	*7.24*	*24.70*	*34.57*	*33*	*27*
Carpentry, rough	...	...	...	...	...	...	...			...	...	...	...	...	...	...	...	...	...
Carpentry, finish	16.6	14.3	13.4	13.2	6.54	15.75	13.6			24.4	16.8	17.5	29	15.1	...	19.9	30.8	...	...
Hardware, rough	...	...	...	...	...	...	...			...	...	...	...	...	...	...	...	...	...
Hardware, finish	...	...	...	...	...	...	...			...	...	...	...	...	...	...	...	...	...
Marble	...	...	...	...	...	...	...			...	...	...	...	...	...	...	...	...	...
Mosaic	...	...	...	...	...	...	...			...	...	...	...	...	...	...	...	...	...
Glass	...	...	...	...	...	...	...			...	...	...	...	...	...	...	...	...	...
Slate	...	...	...	...	...	...	...			...	...	...	...	...	...	...	...	...	...
Plastering	3.32	5.5	3.3	2.66	1.93	5.2	2.74			...	12.6	2.0	6.7	4.3	1.54	0.51	...	...	...
Fresco	...	...	...	...	...	...	...			...	...	...	...	...	...	...	...	...	...
Paint and varnish	2.9	1.65	4	2.33	2.63	2.6	3.71			1.58	5.4	3.26	3.6	2.8	0.30	2.86	2.34	...	...
Office-partitions (wood and glass)	...	...	...	...	...	...	...			...	...	...	...	...	...	...	...	...	...
Ornamental iron	...	...	...	...	...	...	...			...	...	...	...	...	...	...	...	...	...
Skylights and sheet metal	2.05	0.66	1.7	1.15	1.06	1.18	1.2			2.39	1.52	3.6	2.05	2.72	5.4	1.43	1.43	...	...
Office-grill	...	...	...	...	...	...	...			...	...	...	...	...	...	...	...	...	...
General expenses																		[illegible]	[illegible]

Classified construction	Union trust Building	Calvert Building	Herald Building	Continental Building	Equitable Building	Mer Nat Bank Building	Maryland Trust Building	C & P Tel Co Building	Average for eight buildings
	5 62	4 37	7 25					4 3	5 5
	1 22	1 25	2 07					4 3	2 21
		1 6							1 6
	4 4	0.74	5						3.38
		0 65							0 65
		0.13	0.18						0.155
Steel frame	*13 6*	*14 34*	*19 52*	*9 9*	*9 00*	*10.1*	*11 4*	*17 5*	*13.105*
Material		11 79							11 79
Erection		1.7							1 7
Shop-drawings		0 49							0.49
Painting		0.24							0 24
Teaming		0 32							0.32
Masonwork	*23.76*	*28 50*	*34 15*	*30.15*	*31.31*	*37 2*	*29 94*	*35 5*	*31 31*
	10.1	6 59	11 21				8 96	13 5	8 25
		1 82	1.26						1.54
		0 86							0 86
ting		0.21							0 21
	4.34	7 61	5 07	3 9	2 5		3 5		4.48
	2.55	4.22	9.34	4 05	7.4	27.3	4 7	9 4	9 12
					0.83				0.83
	0 77				1.8				1 28
	3.87	4 73	3.68	6 8	6 6	3 4	3 6	6 6	4 91
ver floor-arches	0.57	0 58	0.49					6 0	1.91
	1.56	1 88	3.1						2.18
wrecking									
				7 6			5 16		6 38
and wrecking					0 48				0 48
				7 8	11 70	6.5			8.66
Equipment	*19 18*	*24 44*	*15 06*	*18 60*	*26 71*	*15 23*	*21 23*	*12 60*	*19 27*
Elevator plant	7 6	7 46	8 15	5	5 2	3 8	7 7	4 02	6 15
Plumbing	3 26	3 55	2 61	4 [illegible]	3 5	3 7	3 4	3 [illegible]	3 43
Heating system	4 6	7 06	3 44	4 [illegible]	3 6	4 2	[illegible]	4 15	4 1

The figures opposite each item represent percentages of total cost of building

Classified construction	Union Trust Building	Calvert Building	Herald Building	Continental Building	Equitable Building	Mer Nat Bank Building	Maryland Trust Building	C & P Tel Co Building	Average for eight buildings
					1.9				1.9
and fixtures	2.54	5.05	1.38	2.03	1.9	1.6	3	1.16	2.33
, etc.					2.18				2.18
	0.7	0.49		1.05	0.65				0.72
	0.48	0.27	0.28	0.18		0.33	0.43		0.32
		0.44							0.44
				0.93					0.93
Safes					1.9				1.9
Vault-doors and safe-doors				0.63		3.6			2.11
Flash-signals and indicators				0.54					0.54
Turkish baths					0.68				0.68
Furniture					5.4				5.4
Piping, high pressure					0.70				0.70
Trim and finish	*36.06*	*27.56*	*22.83*	*36.20*	*28.78*	*36.63*	*26.89*	*25.04*	*29.99*
Carpentry, rough	2.03	2.59	0.22	10.9	9.8	5.4	7.9	1.6	5.05
Carpentry, finish	5.27	4.31	0.51					5.7	3.94
Hardware, rough	0.32	0.29	1.7		0.64	0.9	1.16	1.95	0.99
Hardware, finish	0.74	0.88	5.1	0.83					1.88
Marble	6.74	7.11	1.42	10.5	5.4	10.1	6.2	2.8	6.26
Mosaic	0.14		1.99			1.2			1.11
Glass	1.44	9	1.13	1.4	7.5	1.4		0.89	1.23
Slate									
Plastering	2.94	3.05	2.96	2.9	4.21	3.7	2.7	1.4	2.98
Fresco	0.88								0.88
Paint and varnish	2.00	1.34	1.45	2.9	1.6	1.4	2.9	1.5	1.88
Office partitions (wood and glass)	2.65								2.65
Ornamental iron	9.70	5.87	4.62	6.4	5.0	10.5	5.5	6.8	6.79
Skylights and sheet metal	0.71	1.22	1.54	0.37	0.63	0.83	0.53	2.4	1.02
Office-grill	0.5		0.19			1.2			0.63
General expenses	*1.83*	*1.92*		*5.17*	*4.31*	*0.36*	*11.2*	*6.94*	*4.53*
Miscellaneous	1.83	1.92		0.27	0.67	0.36	10	1.94	2.43
Architects' fees, etc.				4.9	1.82			5.0	3.91

Costs * of Different Kinds of Work per Cubic Foot of Building

Some estimates † have been made by F. W. Fitzpatrick showing the prop tionate COST OF THE DIFFERENT BRANCHES OF WORK which go to make up completed building. Believing that these data will be found useful in mak up approximate estimates, Mr. Kidder obtained permission to use them in th Pocket-Book. The following figures represent the actual cost of a TEN-STO OFFICE-BUILDING, 60 by 130 ft in plan, built in the Middle West, a first-cl fire-proof structure, with two street-fronts faced with granite and resting on pile foundation.

Kind of work	Per cubic foot of entire building, cents	Kind of work	Per cubic foot of entire building, cents
Foundations	1 3/4	Heating	1 1/3
Steel framing	2 1/2	Plumbing	1/2
Granite and all masonry	11 1/6	Elevators	1
Cornice, roofs and sky-lights	2/3	Stairs, scenic structural framing, "making ends meet," lamp-fixtures, etc. What might be called a fair amount for "contingencies" in such a building, including lesser items not mentioned here but grouped together	4 23/30
Fire-proof floors	2/3	Architect's fee	1 2/3
Partitions, tile	2/5		
All plastering and stucco	1 1/4		
Elevator-fronts and all ornamental metalwork	2		
Marble work	3 1/6		
Hardware	2/15		
Joiners' work	1 1/6		
Glass	5/12		
Painting and varnishing	7/30		
Electric wiring	2/5	Total	34 5/12

The Chicago post-office building, containing 12 000 000 cu ft and of mon mental character and finish, cost,* in some of its items, as follows:

Kind of work	Per cubic foot of entire building, cents	Kind of work	Per cubic foot of entire building, cents
Foundations	1 3/4	Ornamental metalwork	2 1/4
Steel framing	2 1/2	Marble	5 2/3
Granite and masonry	13 1/2	Plumbing	1/2
Fire-proof floors	2/3	Heating	1 1/4
Plaster, plain and ornamental	1 7/8		

It will be noticed that the relative cost of several of these items is the sam as in the office-building. The total cost * of this building was 42 1/4 cts per cu f

* These pre-war figures must be increased from 50% to 100%.
† [illegible] in "Fireproof," March, 1903.

Cost * of Buildings per Square Foot

One-Story Buildings of Large Area, such as exposition-buildings, etc., may be estimated almost as accurately by the square foot of ground covered as by the cubic foot of building, as there are few or no interior partitions, and usually no plastering or interior finish.

Iron and Steel Buildings. "Roughly speaking, the cost of one-story iron and steel buildings, complete, is, for sheds and storage-houses, from 40 to 60 cts per sq ft of ground, and for such buildings as machine-shops, foundries, and electric-light plants, that are provided with traveling cranes, the cost is from 60 to 90 cts per sq ft of ground covered." †

Structural Steel. For estimates of cost of structural steel for buildings, see pages 1204 to 1207.

Wooden and Brick Mills and Warehouses. See Chapter XXII, pages to 810.

Exposition-Buildings. The cost ‡ of the World's Fair buildings (Chicago, 1893) per square foot of ground covered, including sculpture and decoration, as follows:

Manufactures and Liberal Arts Building	$1.39
Transportation Building	1.08
Electricity Building	1.69
Machinery Hall	2.12
Agricultural Building	1.44
Administration Building	9.18
Horticultural Building	1.41
Mines and Mining Building	1.04
Fisheries Building	2.35
Forestry Building	0.75

Cost * of Structures for the St. Louis Exposition (1904). The following figures were issued by Isaac S. Taylor, at that time Director of Works, of the

Building	Dimensions, ft	Area, acres	Total cost	Cost, sq ft
Art	161×346	1.42	$967 833.90	$5.45
Two Art Pavilions, each	144×423	3.14		
Art Building Annex	106×150	0.41	39 388.99	2.48
Government Building	200×736	3.86	328 980.00	2.23
Government Fisheries	136×136	0.42	45 000.00	2.43
Mines and Metallurgy	525×750	9.08	488 848.50	1.24
Liberal Arts	525×750	8.80	471 820.95	1.20
Education and Social Economy	525×758	7.70	323 950.75	0.81
Manufactures	525×1 200	13.47	711 510.00	1.13
Electricity	525×758	6.67	408 531.57	1.03
Varied Industries	525×1 200	10.28	704 067.96	1.12
Machinery	525×1 000	9.48	509 110.50	0.97
Steam, Gas and Fuel	301×326½	2.25	135 480.00	1.38
Transportation	525×1 300	15.70	674 853.42	0.99
Horticulture	374×782	5.42	225 342.27	0.77
Agriculture	500×1 600	18.62	520 491.07	0.58
Forestry, Fish and Game	300×600	4.07	168 883.38	0.94
Festival Hall	195 in diameter, exclusive of annex	1.09	215 899.00	....

* These pre-war prices must be increased from 50% to 100%. † H. G. Tyrell.
‡ Given by E. C. Chankland, chief engineer.

World's Fair, showing the area and cost of the principal exhibition-bu[illegible] The total area of twenty-two buildings was 123.51 acres, and the to[illegible] $6 939 992.26. The cost was for the bare buildings, and did not include[illegible] tural or other decorations, or the architects' compensation.

Recent Exposition Buildings. The cost of buildings of this c[illegible] erected since 1904, shows a pretty general increase up to 1914, with oc[illegible] variations in the rate of change, of from 1 to 2% per year. Increase si[illegible] would be from 50 to 100%.

Cost * of United States Government Buildings. There was pub[illegible] 1900, by the United States Treasury Department, a history of the [illegible] buildings of the United States, giving their cost, and in 1902, there w[illegible] lished † a list of 287 buildings, giving the cost per cubic foot, the m[illegible] used for the walls and the date of erection. There was also published, in 1[illegible] by the Committee on Public Buildings and Grounds of the United States Se[illegible] a list of sites and plans for public buildings, giving data of much value in [illegible] to the cost of public buildings, their cubical contents and their cost per c[illegible] foot, including buildings erected from 1816 to 1910. "As a rule, these bu[illegible] have cost more per cubic foot than private buildings, so that their cost ca[illegible] always be used as a guide, except for government buildings." ‡

Unit Prices * per Cubic Foot for Recent Government Buildings of [illegible] Same Type.§ The data included in the following paragraphs relate to [illegible] buildings erected before or in process of construction in 1914. They a[illegible] certain FIXED TYPES and in different parts of the United States. The bu[illegible] are post-office buildings and the location, brief description of the general c[illegible] struction, ground-area covered, cubical contents and comparative rates [illegible] cubic foot are given. The buildings are grouped under five different types [illegible] the VARIATIONS IN COSTS PER CUBIC FOOT of similar or identical buildings in [illegible] type, located in different sections of the country, are shown. Following [illegible] five types is a list of buildings of various sizes and descriptions showing [illegible] variations in the cubic-foot rates. The conclusions arrived at and summa[illegible] at the end of the lists, include a table which shows what was considered by [illegible] office of the Supervising Architect to be a fair DIFFERENCE IN COST OF BUIL[illegible] OF THE SAME TYPE in different sections of the United States. It was consid[illegible] also, by that office, that the method of estimating the cost of buildings [illegible] CUBIC-FOOT UNIT PRICE is productive of very uncertain results, inasmuc[illegible] there are many variable conditions entering into the construction of buil[illegible] located in different localities. The principal items affecting the cost of si[illegible] types of buildings are:

(1) Labor; rates and efficiency.

(2) Materials; quality and freight-rates.

(3) Season; time of year when building is constructed.

(4) Contractors; finances, ability, equipment, overhead expenses and ra[illegible] of profit desired.

* Pre-war figures must be increased from 50% to 100%.

† Published in the Architects' and Builders' Magazine, Aug., 1902, and in the In[illegible] Architect, April, 1902.

‡ F. E. Kidder, in previous editions of the Pocket-Book.

§ The information relating to the cost of recent government buildings of certain [illegible] was furnished by J. W. Ginder, Superintendent of the Computing Division, Of[illegible] the Supervising Architect, by permission of Mr. O. Wenderoth, the Supervising A[illegible] tect, through whose courtesy and valuable assistance the editor is able to present [illegible] data referred to. The editor regrets that limited space prevents the reproduction [illegible] carefully prepared and most interesting series of photographs of the plans, eleva[illegible] and sections of the government buildings, the costs of which per cubic foot are discussed.

:) Location; as to supply-centers, distance from railroads, and facilities for dling mtaerials.

'ariations in Unit Costs * of Identical Buildings in Different Localities

order to compare the costs of identical buildings, with slight modification ', the following are given as examples, to show the variance in different lities.

ype 1. Post-office buildings at Grenada, Miss., Bennettsville, S. C., Covon, Tenn., and Burlington, N. J.

escription. Main building, two stories and basement; rear projection, one y and basement; non-fire-proof construction throughout; brick facing; stone ; wooden cornice; slate-covered gable roof, with dormers over two-story ion, and flat, composition roof over one-story portion.

Area and contents	
round-area	3 825 sq ft
ubical contents	138 210 cu ft

Rate per cubic foot *		
Location	Non-fire-proof	First floor, fire-proof
renada, Miss	$0.322	$0.327
ovington, Tenn.	0.315	0.324
ennettsville, S. C.	0.304	0.309
urlington, N. J.	0.293	0.298

ype 2. Post-office buildings at Winchester, Tenn., McPherson, Kan., and ngview, Tex.

Description. Main building, two stories; rear projection, one story; partly avated basement; non-fire-proof construction throughout; brick facing; ne trim; wooden cornice and pilasters at front entrance; slate-covered gable f with dormers over two-story portion, and flat, composition roof over onery portion.

Area and contents	
round-area	3 825 sq ft
ubical contents	138 210 cu ft

Rate per cubic foot *		
Location	Non-fire-proof	First floor, fire-proof
Vinchester, Tenn.	$0.344	$0.350
IcPherson, Kan.	0.346	0.351
ongview, Tex	0.332	0.337

* These pre-war figures must be increased from 50 to 100%.

Type 3. Post-office buildings at Cookeville, Tenn., and Jackson, Ky.

Description. Three-story-and-basement building; stone-faced to top course over water-table; selected, common-brick facing and ornamental terra-cotta trim; composition and slate roof and non-fire-proof construction, except the first floor.

Area and contents	
Ground-area	4 942 sq ft
Cubical contents	290 300 cu ft
Rate per cubic foot *	
Cookeville, Tenn	$0.275
Jackson, Ky	0.269

Type 4. Post-office buildings at Garden City, Kan., and Lake City, Minn. (identical buildings).

Description. One-story-and-basement, brick-faced building, with stone water-table course and trimmings and ornamental terra-cotta cornice, architrave and parapet-coping; non-fire-proof construction, except the first floor; composition roof.

Area and contents	
Ground-area	3 888 sq ft
Cubical contents	141 456 cu ft
Rate per cubic foot *	
Garden City, Kan	$0.405
Lake City, Minn	0.341

Type 5. Post-office buildings at Abilene, Kan., and Bellefontaine, Ohio.

Description. One story and basement; stone facing; granite steps, etc.; tin roof; fire-proof construction, except roof.

Area and contents	
Ground-area	5 000 sq ft
Cubical contents	183 000 cu ft
Rate per cubic foot *	
Abilene, Kan	$0.359
Bellefontaine, Ohio	0.367

Buildings of Various Sizes and Descriptions. The following list is of buildings of various sizes and descriptions throughout the country and shows the variance in the cubic-foot rate.

* These pre-war figures must be increased from 50 to 100%.

ost-office building at New Rochelle, N. Y.

escription. This building is of an irregular plan; two-story and basement; er pavilion; sides and rear one-story and basement; clearstory over work-n; stone facing to first-floor level; brick facing above this point, with terra-a trim and cornice; composition roof; fire-proof construction.

Ground-area	7 512 sq ft
Cubical contents	258 900 cu ft
Rate per cubic foot *	$0.259

ost-office building at Mobile, Ala.

escription. Front portion, two stories, and rear portion, one story over kroom. Only a small portion of basement excavated for heating-plant. n building faced with limestone and rear second story portion with orna-tal terra-cotta. Fire-proof construction; long and short spans, and con-e joists with terra-cotta fillers; copper deck and Spanish-tile roofs.

Ground-area	18 054 sq ft
Cubical contents	670 476 cu ft
Rate per cubic foot *	$0.341

ost-office building at Muskogee, Okla.

escription. A four-story-and-basement building. Granite to the first-line, stone-faced above (except in interior court, which is brick); terra-a cresting at roof; copper roofing and fire-proof construction throughout. n standard types of concrete and terra-cotta floor-construction. Monu-tal in design. Corinthian colonnade at entrance. Eight heavy bronze -standards. Six flights of marble stairs. Entire lobby of marble, and ornamental plaster-work in lobby and court-room.

Ground-area	20 400 sq ft
Cubical contents	1 326 612 cu ft
Rate per cubic foot *	$0.43

ost-office building at New Bedford, Mass.

escription. One story, basement and mezzanine with clearstory over ral portion; granite facing, except clearstory, which is faced with terra-a; main roof of composition; clearstory roof of copper; fire-proof con-ction.

Ground-area	27 750 sq ft
Cubical contents	1 080 690 cu ft
Rate per cubic foot *	$0.323

ost-office building at Newark, Ohio.

escription. Two-story, basement and unfinished attic. The workroom ex-s through two stories. Offices in second story over balance of building. proof construction throughout. Terra-cotta floors, ceilings, roofs, parti-, furring, etc. Exterior faced with pink granite to the first-floor level and white marble above, including cornice, parapet, etc. Flat tin roof; bronze s at first and second-story windows on front of building. Cast-iron grilles rst-story and basement-windows on sides and rear; bronze-faced post-screens, desks, revolving doors, vestibules, etc., and drawn-bronze covered s, window-frames, doors, etc., in lobby. Caen-stone cornice and coffered g in lobby. Bronze and marble stairs to second story.

* These pre-war figures must be increased from 50 to 100%.

Ground-area	6 912 sq ft
Cubical contents	369 640 cu ft
Rate per cubic foot *	$0.487

Post-office building at Minot, N. D.

Description. Three-story-and-basement building; fire-proof, except r which is plank on steel beams; stone facing to second-story window-sills; b facing above, with stone cornice, parapet-coping, etc.

Ground-area	6 700 sq ft
Cubical contents	427 300 cu ft
Rate per cubic foot *	$0.328

Post-office building at McAlester, Okla.

Description. Three stories and basement; fire-proof, except roof; te cotta floors, etc.; suspended ceilings; stone facing to second-floor level; b facing above, with stone trim; cornice and balustrade; tin roof.

Ground-area	7 482 sq ft
Cubical contents	394 765 cu ft
Rate per cubic foot *	$0.38

Post-office building at North Tonawanda, N. Y.

Description. The building has two stories and basement; granite to first-floor line; brick-faced above with stone trimming and slate roof; fire-p construction to and including the second floor.

Ground-area	5 475 sq ft
Cubical contents	276 320 cu ft
Rate per cubic foot *	$0.289

Conclusions Regarding Variations in Unit Costs. In the foreg unit costs, the APPROACH-WORK, such as walks, platforms, terraces, etc., is cluded. This, in some cases, is quite expensive, and is generally from 5 to of the entire cost of the building. In federal buildings, there are many requ ments not met with in the ordinary mercantile buildings, and the perma character of the building necessitates all materials, workmanship and const tion to be of the very best in each case. This is guaranteed by iron-clad sp fications, long-time guarantees for several items of the work, and pers government inspection. The office of the supervising architect has de mined that the RELATIVE INCREASE in cost of buildings throughout the cou over the cost in the Mississippi Valley district was about as follows, taking Mississippi Valley district, as a BASE, at 100%, and the labor and market-c ditions which prevailed in October, 1914.

	Per cent
Mississippi Valley district	100
New England (except Maine)	110
Maine	115
Southern States	100
Northwest Mountain district	130
Southwest Mountain district	120
Pacific Coast	125

* These pre-war figures must be increased from 50 to 100%.

In the grouping of districts, the Mississippi Valley district is intended to ver the Middle States as far east as Ohio and Pennsylvania, and the states, erally, bordering on the western bank of the Mississippi River. This is nd to be a part of the country in which the LOWEST PRICES have been obned. The other districts represent the approximate greater cost for buildings r that in the Mississippi Valley or Middle States, and is intended to repret the DIFFERENCE IN COST AT ANY TIME; but is not intended to represent difference in cost at different periods.

Illustration of Variation in Cost * of Buildings of Identical Area and ntents. The following notes are taken from photographs of drawings and n data accompanying them.† The drawings were for a Post-Office build- at Menomonie, Wis. This building contains 4 770 sq ft of ground-area, the cubical contents are 147 570 cu ft. The contract was awarded (1913) $45 380, or at the rate of $0.308 per cu ft. It is a one-story-and-basement lding, faced with brick, with stone water-table, brick parapet and tin and iposition roof. The first floor, only, is fire-proof. Proposals were opened 14) for a Post-Office building at Uvalde, Tex. This building, except for e slight modifications, is as nearly like the Menomonie building as it is sible to make it without using the same drawings. The ground-area of the lde building is 4 672 sq ft and the cubical contents, 151 875 cu ft. The work onnection with the approaches is practically the same as that at Menomonie. hese buildings had been erected in the same town, it does not appear that re would have been any difference in the costs, but the lowest proposal ived for the Uvalde building was $56 400, or at the rate of $0.371 per cu ft. omparison of the amounts for these two buildings further illustrates the eliability of any universal application of the cubic-foot rate in determining costs of buildings, and also shows that the difference in cost of construction uildings in different sections of the country varies considerably.

ost per Cubic Foot of Some Important Federal Buildings. The follow- tabulations contain additional unit costs and other data for public buildings.

Cost * per Cubic Foot of Some Important Federal Buildings.

Location and building	Cost per cubic foot, cents *
w York, N. Y., Custom-House (completed 1908)	74
eveland, Ohio, Post-Office, Custom-House and Court-House	68
n Francisco, Cal., New Post-Office and Court-House (completed 1906)	66
nver, Col., new Mint (completed 1905)	65
n Francisco, Cal., Subtreasury Building (estimated)	60
ltimore, Md., new Custom-House (completed 1908)	55
shington, D. C., Senate Office-Building	50
lt Lake City, Utah, Post-Office (completed 1905)	47
dianapolis, Ind., new Post-Office (completed 1906)	46
iladelphia, Pa., new Mint (completed 1901)	45
shington, D. C., National Museum Building	43
shington, D. C., Agricultural Buildings (portions completed)	40
shington, D. C., House Office-Building	36

These pre-war figures must be increased from 50 to 100%.
These photographs of plans, elevations and sections, together with many others, and mpanying explanations and data, were furnished the editor by J. W. Ginder, Superdent of the Computing Division, Office of the Supervising Architect, by permission r. O. Wenderoth, the Supervising Architect (1914), and have been of great assistance e presentation of notes on the costs of buildings.

Cost * per Cubic Foot and per Square Foot of Some New Public Buildings†

Location	Facing	Cost	Contents, cu ft	Area, sq ft	Cost	
					Cu ft	Sq ft
Bangor, Me	Granite	$271 297	793 720	15 600	$0.342	$17.4
Augusta, Ga	Marble	288 800	576 000	11 000	0.500	26.2
South Chicago, Ill	Stone	132 702	377 668	11 000	0.350	12.0
Long Branch, N. J	Limestone	95 200	256 210	6 470	0.373	14.7
Plymouth, Mass	Brick	81 532	256 210	6 470	0.318	12.6
Piqua, Ohio	Limestone	116 689	448 300	9 984	0.360	11.7
New Bedford, Mass	Granite	295 051	1 080 000	21 732	0.300	13.5

Depreciation of Buildings ‡

Discounts from Values of New Buildings. The figures given on the preceding pages are for new buildings. To ascertain their value at any time subsequent to their erection, a discount from the value when new should be made as follows:

	Per cent per year
Brick, occupied by owner	1 to 1¼
Brick, occupied by tenant	1¼ to 1½
Frame, occupied by owner	2 to 2½
Frame, occupied by tenant	2½ to 3

If built of long-leaf yellow pine, or of spruce from the New England States, add from 20 to 30%, or if of short-leaf yellow pine, add from 40 to 50% to the values. If of redwood or cedar from the Pacific Coast, use about one-half these estimates, which are for white pine or white pine with oak framing-timbers. These figures for depreciation are to include buildings in which ordinary repairs have been made. If extraordinary repairs have been made, the discount should not be so heavy. Good judgment must be used in estimating the amount of depreciation in buildings.

The Depreciation of Mill-Buildings. The annual depreciation of a mill-building of slow-burning construction varies from 1 to 1½%, while the depreciation of a reinforced-concrete factory-building is relatively much less, since it is confined entirely to such details as windows, doors, roofing, etc.

The Wear and Tear of Building Materials. At the tenth annual meeting of the Fire Underwriters' Association of the Northwest, held at Chicago, September, 1879, Mr. A. W. Spalding read a paper on the wear and tear of building materials and tabulated the results of his investigations in the following form:

* These pre-war figures must be increased from 50 to 100%.

† Reproduced, by permission, from the Journal of the Society of Constructors of Federal Buildings, September, 1914, through the courtesy of C. R. Marsh, Editor Publications of the Society of Constructors of Federal Buildings. This Journal, published monthly, contains data of much interest to architects and builders.

‡ From Tiffany's Estimate of Depreciation, used by the United States Government

	Frame dwelling		Brick dwelling (shingle roof)		Frame store		Brick store (shingle roof)	
laterial in building	Average life years	Depreciation per annum %	Average life years	Depreciation per annum %	Average life years	Depreciation per annum %	Average life years	Depreciation per annum %
3rick	....	......	75	1⅓	....	......	66	1½
'lastering	20	5	30	3⅓	16	6	30	3⅓
'ainting, outside	5	20	7	14	5	20	6	16
'ainting, inside	7	14	7	14	5	20	6	16
ihingles	16	6	16	6	16	6	16	6
;ornices	40	2½	40	2½	30	3⅓	40	2½
Veather-boarding	30	3⅓	....	......	30	3⅓	....	......
iheathing	50	2	50	2	40	2½	50	2
'looring	20	5	20	5	13	8	13	8
)oors, complete	30	3⅓	30	3⅓	25	4	30	3⅓
Vindows, complete	30	3⅓	30	3⅓	25	4	30	3⅓
tairs and newels	30	3⅓	30	3⅓	20	5	20	5
ases	40	2½	40	2½	30	3⅓	30	3⅓
nside blinds	30	3⅓	30	3⅓	30	3⅓	30	3⅓
uilding hardware	20	5	20	5	13	8	13	8
'iazzas and porches	20	5	20	5	20	5	20	5
utside blinds	16	6	16	6	16	6	16	6
ills and first-floor joists	25	4	40	2½	25	4	30	3⅓
)imension-lumber	50	2	75	1⅓	40	2½	66	1½

'hese figures represent the averages deduced from the replies made by eighty-ee competent builders unconnected with fire-insurance companies in twenty-en cities and towns of the eleven Western States.

THE QUANTITY SYSTEM *

:xplanation of the System. The QUANTITY SYSTEM is not, as some persons e supposed, merely the taking off of a list of items by one person probably h uncertain accuracy, for some other person's use. It means the careful isurement by a disinterested expert specially trained in this kind of work, t is, a QUANTITY SURVEYOR. This specialist proceeds in a manner quite erent from that of the average contractor. He follows a certain recognized er and system in taking off quantities, abstracting and billing, with a view liminating errors. He uses certain uniform standards of measurements and ressions well understood by bidders. His checking and rechecking methods ensure accuracy must be studied to be appreciated by those to whom the ntity system is unknown. A record is kept of every item, however small, ing a money-value. These items are classified and arranged, each under proper trade or department, in methodical order. Guess-work methods

The quantity "system" which is not merely a survey of items, has been systematically ocated since 1891 by G. Alexander Wright, A.I.A., 354 Pine Street, San Francisco, is the founder of the movement to adapt the Quantity System to American building tice. It has attracted much attention among contractors, architects, and engineers. ourse of time this system of estimating must be adopted, as it stands for a square between owner and contractor. The movement in aid of this work is purely a ntary one, an honest effort to bring about better methods.

are unknown to the quantity surveyor, while his accuracy and attention to small details is worthy of comment. Every bidder figures from a copy of the surveyor's quantities furnished to each one, with (if desired) the plans and specifications. The surveyor who does this work is a professional man similar to the engineer or the architect. He should, in fact, have, and he usually has had, experience in these professions, and in addition, a practical experience acquired in the field in actual contact with and superintendence of construction work.

Method of Procedure. Such a surveyor, in taking off quantities from the architect's or engineer's drawings, readily detects any discrepancies due to hasty preparation or other cause. The attention of the architect or engineer is called to such matters by the quantity surveyor, as he goes on with his work. Detected in this way, all uncertainties are at once corrected and adjusted, so that by the time the drawings and specifications reach contractors, everything has been made plain and accurate and the possibility of error in quantities can therefore be disregarded. The resulting document, the BILL OF QUANTITIES is then either printed or otherwise reproduced, and a facsimile copy supplied free of cost to each bidder who inserts his unit price opposite each item and in an hour or two foots up the money-cost in dollars and cents. This is really all that a contractor should be expected to do (for nothing). The BILL OF QUANTITIES contains everything the contractor is called upon to perform or furnish, in order to complete his contract. In short, the bid becomes a proposal to do a certain FIXED QUANTITY of work, no more and no less. This then, briefly, is the main underlying principle of the QUANTITY SYSTEM: a definite quantity of work for a definite price, and the elimination of every condition which now compels bidders to take chances.

The Present Unsatisfactory Conditions. Most architects are familiar with the wasteful, unsatisfactory methods followed to-day. They injure both parties to a contract because of bidders' mistakes in figuring, accuracy being so often sacrificed for speed. While wonderful strides in methods of construction have been made, no attention has been given to STANDARDIZING METHODS of measuring builders' work, and so both owner and contractor suffer. As a result of the movement in aid of better methods (initiated in San Francisco in 1891) more conservatism, and a closer adherence to business principles is being preferred in place of gambling methods of estimating. Architects and engineers who now permit an unduly low bidder to take a contract are courting trouble every time.

Use of the Quantity System in Other Countries. The principle of payment by measurement is based upon equity and square dealing. On large work it is used in England, Ireland, Scotland, France, Germany, Australia, and South Africa, and to some extent in the United States and Canada. It is a significant fact, that in no instance in which this measurement system has been once established, has it ever been abandoned for the former haphazard methods.

Advantages Claimed for the Quantity System. The following are the advantages claimed for the system:

(1) An immense saving of time and money now wasted by bidders, all doing the same thing, going over the same ground, and each arriving at a different result.

(2) Safer bids, as the work to be performed is clearly written out in the bill of quantities, which can be the essence of the contract.

(3) No expense to the bidder; the owner pays for the quantities knowingly, ie owners pay now, but this fact is not brought to their attention, and it es not occur to them. The percentage added to a bidder's net cost is not profit, a certain portion being absorbed in overhead charges, including cost estimating, which, of course, is ultimately borne by owners.

(4) Saving of disputes arising from ambiguities, oversights, and even errors, causing extra claims more or less just, but usually vexatious, and sometimes ibarrassing.

(5) Better opportunities for the competent bidder, as the bidders all work and price from the same basis.

(6) Better work and greater harmony. If no part of the work is omitted ere is less reason to skin the work, a proceeding which produces friction, or irse.

(7) Misunderstandings are reduced. The bill of quantities states clearly iat is intended, and is a sort of clearing-house for the drawings and specifica-ns.

(8) Neither party can obtain an advantage over the other on quantity or scription of work.

(9) No disputes with subbidders, it being clearly stated what each trade is furnish.

(10) Contractors have no figuring of quantities to do and can therefore vote more time to buildings in hand and save profits now lost for want of eir personal supervision.

(11) Fewer inferior contractors as lowest bidders.

(12) Fewer extras, which are usually a trouble to all concerned.

(13) The architect or engineer has the assistance by collaboration of the ofessional quantity surveyor, who is available, also, for preliminary figures. is advance-information, now so often furnished by a prospective bidder, eates undesirable obligations.

(14) No change or reorganizing of architects' offices is entailed. Much tail-work now involved in receiving bids could be taken care of in the antity surveyor's office.

(15) The drawings and specifications having been previously made as com-te as possible, subsequent inconvenience to contractors and foremen on the , and inquiries at the architects' offices for explanations become unneces-y. The BILL OF QUANTITIES gives detailed information which cannot be well en by drawings.

Adaptation to American Practice. In the United States any such uni-rsal system must conform to American needs and sentiment, and be a prac-al system. For many reasons it would be unpractical to follow the English ctice. The principles it stands for can, however, be accepted and applied ywhere with great advantage.

IMENSIONS AND DATA USEFUL IN THE PREPARA-TION OF ARCHITECTS' DRAWINGS AND SPECIFICATIONS*

Dimensions for Furniture. For the convenience of draughtsmen when igning furniture or providing space for a special article the following dimen-ns are given: †

* See, also, the additional tables with more detailed and classified lists.

† Many of these dimensions were first contributed to the American Architect of No-nber 10, 1894, by Alvin C. Nye.

Chairs and Seats. The average figures taken from a variety of good ch are: Height of the seat above the floor, 18 in; depth of the seat, 19 in; the h of the back above the floor, 38 in. Usually the seat increases in depth a decreases in height, while the back is higher and slopes more. Twenty inch inside is a comfortable depth for a seat of moderate size. Chair-arms are ab 9 in above the seat. The slope of the back should not be more than one-t the depth of the seat. A LOUNGE is 6 ft long and about 30 in wide.

Tables vary in shape and size almost as much as chairs. Writing-tables a dining-tables are made 2 ft 5 in high, and the type of sideboard called a CARD TABLE is made 3 ft high to the principal shelf; but tables for general use are 2 6 in high. DINING-TABLES are made from 3 ft 6 in to 4 ft wide and to ext from 12 ft to 16 ft by means of slides within the frame. This frame should n be so deep as to interfere with the knees of any one sitting at the table; that i there must be about 2 ft clear space between it and the floor. The smallest s practicable for the KNEE-HOLES of desks and library-tables is 2 ft high by 1 ft 8 wide, the width to be increased as much as possible.

Bedsteads are classed as SINGLE, THREE-QUARTERS, and DOUBLE. A sing bed is from 3 to 4 ft wide inside; a three-quarter bed, from 4 ft to 4 ft 6 i a double bed, 5 ft. Bedsteads are from 6 ft 6 in to 6 ft 8 in long inside. Fo boards are from 2 ft 6 in to 3 ft 6 in and headboards from 5 ft to 6 ft 6 in hig Single beds for dormitories are often made only 2 ft 8 in wide.

Bureaus vary in shape and size to such an extent that it is almost impossi to say that any dimension is fixed. Convenient sizes are: body, 3 ft 5 in wid 1 ft 6 in deep and 2 ft 6 in high; or 4 ft wide, 1 ft 8 in deep and 3 ft high.

Commodes are 1 ft 6 in square on the top and 2 ft 6 in high.

Chiffoniers are about 3 ft wide, 1 ft 8 in deep and 4 ft 4 in high.

Cheval-Glasses are made, if large, 6 ft 4 in high and 3 ft 2 in wide. If smal 5 ft high and 1 ft 8 in wide. If medium, 5 ft 6 in high and 2 ft wide.

Wash-Stands of large sizes are 3 ft long, 1 ft 6 in wide and 2 ft 7 in high. Sma sizes are from 2 ft 4 in to 2 ft 8 in long.

Wardrobes may be 8 ft high, 2 ft deep and 4 ft 6 in wide; or 6 ft 9 in hig 1 ft 5 in deep and 3 ft wide.

Sideboards may be from 4 to 6 ft long and from 20 in to 2 ft 2 in deep.

Upright Pianos vary from 4 ft 10 in to 5 ft 6 in in length, from 4 to 4 ft 9 i in height and are about 2 ft 4 in deep over all.

Miniature and Baby-Grand Pianos vary from 5 ft 10 in to 6 ft in length, a are about 4 ft 10 in in width.

Parlor-Grand Pianos vary from 5½ ft to 6 ft 10 in in length, and are abo 4 ft 10 in in width.

Concert-Grand Pianos are about 8 ft 10 in in length and 5 ft in width.

Billiard-Tables (Collender), 4 by 8 ft, 4 ft 2 in by 9 ft and 5 by 10 ft. Si of room required 13 by 17 ft, 14 by 18 ft and 15 by 20 ft, respectively.

Classified Tables * of Furniture-Dimensions. The following more d tailed and classified tables of average dimensions of furniture are added to tho already given and are taken from recent data furnished by manufacturers

* These additional tables were compiled by E. S. Hand, and much of this data in t several editions of the Pocket-Book has been taken, by permission, from the valuabl treatise on Furniture Designing and Draughting, by A. C. Nye.

rniture. While some of these measurements vary slightly from the dimen ons given in the preceding paragraphs they represent average dimensions o rniture as made at the present time.

Dimensions of Tables

Kind of table	Length	Width	Height	Remarks
Bedroom-table	31	22	29	....
Bedroom-table	18	18	30	Commode
Bijou-table	30	22	30	....
Carving-table	42	20	36	....
Dressing-table	36	20	30	....
Extension table	66	66	30	Round
Extension table	54	54	30	Square
Library-table	51	41	30	Oval
Library-table	42	27	29	....
Library-table	54	34	29	....
Library-table	60	36	29	....
Tea-table	13	13	20	Round
	18	18	24	Square
	23	17	29	Upper shelf
	30	23	18	Lower shelf

All dimensions are in inches. Heights are from the floor.

Dimensions of Chairs

Kind of chair	Height	Seat-width, Front	Seat-width, Back	Depth, outside	Back Height	Back Slope	Arms, height from floor
Bedroom-chair	18	16	13	17	34	3½	......
Baby's high chair *	20	14	12	13½	37	3	27
Cheek-chair †	17	29	25	27½	44	4½	......
Chip-chair	17	22	17½	17	39	.....	......
Chip-chair	18	22	17	17¾	38	.....	......
Dining-chair	20	24	22	22	45	2½	26½
Dining-chair	20	19	17	19	43	2	......
Dining-chair	19	19	17	18	38½	1½	......
Dining-chair	18	20	15	15	36	2	......
Easy chair	17	33	28	24 ¶	43	5	21
Easy chair †	17	27	25	27½	41	6½	26
Hepplewhite chair	18	21½	17	17	34½	2	27
Parlor-chair ‡	16½	24	19½	18¾	36	4	25¾
Parlor-chair †	14	21	21	18 ¶	29	.....	......
Parlor-chair †	18	26½	22½	26½	37	4	25
Parlor-chair §	18	20	13	19	36	3	23
Piano-bench	20	40	.....	15	......	.....	......
Reception-chair ‖	17	21	19	21	30	2	......
Rocking-chair	16	23½	20½	19½	41	2	24
Roundabout chair	18	18	18	18	29½	0	28½
Rubens chair	20½	17½	17½	15	40	0	......
Slipper-chair	12	18	15	17	28	3	......

* Foot rest 12 in above floor. † Overstuffed. ‡ French cane seat and back. § Wooden arm and back. ‖ Upholstered seat. ¶ Depth inside.

All dimensions are in inches. Heights are from the floor. •The slope of the back is mea ed at the seat-level to a perpendicular through the highest point of the back.

Dimensions of Sofas

Kind of sofa	Height	Seat-width		Depth, outside	Back		Arms, height from floor
		Front	Back		Height	Slope	
Small	18	43	40	21	32½	3	24
Extra large	16	78	76	36	29	2	25
Ordinary sofa	15	54	51	24	34	5½	24
Lounge	17	68	68	28	35	2½	29
Lounge	17	57	57	29	23	12	34

All dimensions are in inches. Heights are from the floor. The slope of the back is measured at the seat-level to a perpendicular through the highest point of the back.

Dimensions of Case-Work

Kind of case-work	Body			Remarks
	Width	Depth	Height	
Bureau	45	20½	36½	....
Bureau	51	23	37½	....
Bureau	48	22	36½	....
Bureau	54	20	42	....
Bookkeeper's desk	60	33	42	....
Bookkeeper's desk	60	32	44	Deck, 11 in; slope, 22 in
Chiffonier	39	20	48	....
Chiffonier	36	20	51	....
Cheval-glass	25	....	65	....
Commode	16	16	31	....
Sideboard	84	32	30	....
Wardrobe	36	19	69	....
Wardrobe	54	24	96	....

All dimensions are in inches. Heights are from the floor. The slope of the back is measured at the seat-level to a perpendicular through the highest point of the back.

Dimensions of Bedsteads

Kind of bed	Inside		Heights		Width, side rail	Height bottom of side rail
	Length	Width	Foot	Head		
Single bed	78	42	40	62	9½	9½
Single bed	78	42	41	60	10	10
Double bed	78	58½	42	63	11	10½
Double bed	78	56	36	67	13	9½

All dimensions are in inches. Heights are from the floor.

Dimensions of Plumbing-Fixtures. Enameled-Iron Bath-Tubs. Standard sizes for roll-rim baths with sloping ends are: nominal lengths, 4 ft, 4½ ft, 5 ft, 5½ ft and 6 ft; width over all, from 30 to 34 in. Specially narrow tubs are made from 25 to 29 in wide. The actual length over rim is usually 1 or 2 in more than the nominal length, and 2 in will include an ordinary overflow-pipe.

Wash-Basins. Crockery basins, to go with marble slabs, are made round and oval. Round bowls are made 10, 12, 13, 14 and 16 in in diam, measured from the outside of the rim. Oval bowls, 14 by 17 in, 15 by 19 in and 16 by 21 in. The 12 and 14-in round, and 15 by 19-in oval, are commonly used.

Marble Basin-Slabs may be 20 by 24 in, 20 by 30 in, 22 by 28 in, or 24 by 30 in, the last being a very common size. They can be made any size, to order. They should be 1¼ in thick, countersunk on top, and should have molded edges where exposed.

Corner-Slabs are commonly made 21 by 21 in and 24 by 24 in. Marble backs are usually 8 or 10 in high, and sometimes 12 in.

Enameled-Iron Wash-Basins or Lavatories made in one piece: common sizes are 16 by 20 in, 11 by 14-in basin; 18 by 21-in, 11 by 15 in basin; 18 by 24 in, 12 by 15-in basin; back, 10½ in high. The smallest-sized wash-basin is 13 in wide at the back.

Corner-Basins, 12½ by 12½ in, 12-in round basin; 15 by 15 in, 11 by 14-in basin; 16 by 16 in, 11 by 14-in basin; 19 by 19 in, 11 by 15-in basin. The standard height of wash-basins is 2 ft 6 in from the floor.

Foot-Baths, enameled iron, roll-rim, are 22½ by 19 in; width, including ittings, 1 ft 11 in; height 17 in; depth inside, 11 in.

Seat-Baths, enameled iron, average about 32 in long over fittings, and 27 in wide.

Water-Closets. The dimensions of water-closet bowls vary considerably, he following being about an average: width of bowl over all, 13 in; depth from vall to front of seat, 23 in; height from floor to seat, 17 in; width of seat, from 5 to 16 in. Closets with low-down tanks measure about 28 in from front of eat to wall. The distance from center of outlet-opening to the walls, or the OUGHING-IN dimensions, are given in manufacturers' catalogues, as they vary vith different closets. The smallest space permissible for water-closet compartments, where doors open out, is 2 ft 4 in by 4 ft. If the doors open in, the compartment should be 3 by 5 ft.

Closet-Ranges, used in schools and factories, are made 24, 27 and 30 in, enter to center of partitions. For graded schools, 24 in is ample, and for ctories, 27 in. The range usually occupies a space 28 in in depth, if set against wall.

Urinal-Stalls should be from 24 to 27 in, center to center of partitions; depth partitions, 20 or 22 in; of ends, 2 ft; of bottom slab, 2 ft; height of partitions, om 4 ft 6 in to 5 ft 6 in.

Kitchen-Sinks of cast iron are made in a great variety of sizes, those most mmonly used being 16 by 24 in, 18 by 30 in, 18 by 36 in, 20 by 30 in and 20 by ; 24 by 50 in is the largest size for enameled sinks. The depth inside, for e sizes given, is 6 in. Plain cast-iron sinks are made as large as 32 by 56 in, or by 78 in. Steel sinks are made in all of the above sizes up to 20 by 40 in.

Porcelain Sinks. Common sizes of porcelain sinks are 20 by 30 in, 23 by 36 in d 24 by 42 in.

Cast-Iron Slop-Sinks, common sizes, are 16 by 16 in, 16 by 20 in, 18 by 22 in d 20 by 24 in; 12 in deep.

Copper Pantry-Sinks. Common sizes are 12 by 18 in, 14 by 20 in and 16 by in.

Laundry-Tubs of slate or soapstone are commonly made 2 ft wide over all, d 16 in deep. Lengths over all, two-part tubs, 4 ft and 4 ft 6 in; three-

part tubs, 6 ft, 6 ft 6 in and 7 ft. Earthen and porcelain tubs come separately and are connected as required. The dimensions of each tub are 2 ft or 2 ft 7½ in in length, 2 ft 1½ in in width and 15 in in depth, inside. The length required for two 2-ft tubs is 4 ft 1 in; for three tubs, 6 ft 2 in; and for four tubs 8 ft 3 in. Wolff's roll-rim enameled-iron wash-tubs are 55 in over all, for two tubs, and 82 in for three tubs.

Range-Boilers are 12 in diameter for 30-gal, 14 in for 40-gal, 16 in for 52-gal and 63-gal, 22 in for 100-gal and 120-gal boilers.

Dimensions of Carriages. Covered Buggy (Goddard). Length over all, 14 ft: width, 5 ft; height, 7 ft 4 in. Will turn in space from 14 to 20 ft square, according to skill.

Coupé. Length over all, 18 ft; width, 6 ft; height, 6 ft 6 in.

Buggy (Piano-Box). Length over all, 14 ft; width, 4 ft 10 in.

Landau. Length over all, 19 ft 6 in; width, 6 ft 3 in; height, 6 ft 3 in; length of pole, 8 ft 0 in.

Stanhope Gig, Two Wheels. Length over all, 10 ft 6 in; width, 5 ft 8 in; height, 7 ft 6 in.

Victoria. Length, without pole, 9 ft 6 in; length of pole, 8 ft; width over all, 5 ft 4 in.

Light Brougham. Length, without pole or shaft, 9 to 11 ft; width over all, 5 ft 4 in; height, 6 ft 4 in.

Automobiles. Length, from 11 to 19 (average 16) ft; width, 6 ft; height, 7 ft.

Dimensions and Weight of Fire-Engines. From measurements of different fire-engines belonging to the city of Boston, it was found that the greatest length, including pole, was 22 ft 6 in. The widths varied from 5 ft to 5 ft 11 in, the average height being 8 ft 8 in. The average weight (computed from 9 engines), 8 000 lb; the greatest weight, 9 420 lb and the least, 4 780 lb.

Dimensions and Weight of Hose-Carriages. Extreme length with horse, 19 ft 6 in, without horse, 17 ft 6 in; width, from 5 ft 9 in to 7 ft; height, from 5 ft 8 in to 7 ft; average weight (computed from 11 carriages), 2 943 lb; greatest weight, 3 500; least weight, 2 120.

Dimensions and Weight of Ladder-Wagons. Length of truck, 33 ft; total length, with ladders on, 45 ft; width, 6 ft 2 in; average weight (computed from 12 wagons), 6 660 lb; greatest weight, 8 800; least, 4 350.

Dimensions of Locomotives and Cars. The dimensions of locomotives and freight-cars vary considerably, but the following will cover those in common use:

Locomotives. From 15 ft 4 in to 15 ft 10 in to top of stack from top of rail; extreme width of cab, 10 ft 2 in. Doors to admit locomotives should be from 12 to 13 ft wide and 18 ft high.

Furniture-Cars are 14 ft 1 in, from top of track to top of brake-staff; floor, 3 ft 8 in from track; extreme width, 9 ft 10 in.

Stock-Cars, 13 ft 5 in, from top of track to top of brake-staff; floor, 4 ft from track; extreme width, 9 ft 8 in.

Refrigerator-Cars, 14 ft 6 in, from top of track to top of brake-staff; floor, 4 ft from track; extreme width, 9 ft 7 in.

Ordinary Freight-Cars are about 13 ft high to top of brake-staff and 9 ft 4 in in extreme width. The height of floor of freight-cars varies from 3 ft 8 in to 4 ft above top of track for STANDARD-GAUGE, and from 3 ft to 3 ft 6 in for NARROW-GAUGE cars. Standard-gauge, 4 ft 8½ in.

Passenger-Coaches vary from 14 to 16 ft in height and from 10 to 11 ft in idth. Doors to admit cars should give at least 12 in clearance on each side, d 2 ft overhead.

Street Trolley-Cars are about 8 ft 6 in wide for the car proper, and the steps oject about 8 in. Height from track to top of coach, 11 ft 6 in; the trolley-ind is 18 in higher. The length varies, up to 42 ft. Trucks for a 41 ft 6 in : are about 24 ft apart. Wheel-bases, 4 ft center to center. Radius of short-: curve in Denver, Colo., 35 ft to midway between rails.

The Gauge of a railroad track is the distance between the inner sides of the ids of the two rails. The STANDARD GAUGE is 4 ft 8½ in

Capacity of Freight-Cars. Car-Loads. The capacity of freight-cars, and : minimum car-loads, vary so greatly that no accurate general information ı be given. For heavy freight, 25 tons is an average load; for light freight, m 12 to 15 tons; for household goods, 10 tons is about the minimum; for .e, 15 tons is about a minimum load; for cement, 20 tons. The minimum -load, to obtain car-load rates, varies with different roads, and also with the e made; a low rate is usually made on the basis of a big load. Thirty tons ı good load for heavy freight, and 40 tons is about the maximum, except for cial cars.

Miscellaneous Dimensions. Horse-Stalls. Width, from 3 ft 10 in to 4 ft else 5 ft or over; length, 9 ft. The width should never be between 4 ft and :. as a horse is liable to cast himself.

Dimensions of Standard Bowling-Alleys.* For ONE PAIR OF ALLEYS: Room essary, 83 ft over all; 11 ft 6 in wide, 60 ft from foul-line to head pin, 3 ft pins to back of alley, 4 ft for pin-pit, 8 in deep in front, 6 in in back; alleys, naple flooring, should extend on and beyond the foul-line 12 ft, and then more, making a 16-ft approach to the foul-line for the player to run to deliver ball. For ONE ALLEY: Same length, 83 ft; width, 6 ft 3¼ in; closer dimen-is; beds 42 in, gutters 9 in, division-pieces 2¾ in, ball-return 9¾ in.

	In
E ALLEY: Ball-return	9¾
st-division piece	2¾
tter	9
l	42
tter	9
ond-division piece	2¾
6 ft 3¼ in =	75¼

	In
ONE PAIR OF ALLEYS: Ball-return	9¾
First-division piece	2¾
Gutter	9
Bed	42
Gutter	9
Second-division piece	2¾
6 ft 3¼ in =	75¼

'o the 75¼ in of the PAIR OF ALLEYS, should be added

ter	9
	42
ter	9
rd-division piece	2¾
	138

dditional room should be provided for the bowlers and spectators as these ensions are for the alleys only.

imensions of Drawings for Patents (United States). 10 by 15 in, with ler-line 1 in inside all around.

* Dimensions furnished by The Brunswick-Balke-Collender Company, New York City.

Dimensions of a Barrel. Diameter of head, 17 in; diameter at bung, 19 length, 28 in; volume, 7 680 cu in.

Miscellaneous Memoranda. Weight of Men and Women. The aver weight per person of twenty thousand men and women weighed at Boston, Ma in 1864, was, men, 141½ lb; women, 124½ lb.

Wooden Flagpoles. For a flagpole, extending from 30 to 60 ft above roof, the following proportions give satisfactory results: The diameter at roof should be 1/60 the height above the roof, and the top diameter one-half lower. To profile the pole, divide the height into quarters; make the diam at the first quarter above the roof, fifteen-sixteenths of the lower diamet at the second quarter, seven-eighths, and at the third quarter, three-quart the lower diameter.*

Steel Flagpoles.† The Department of Education, City of New York, h abandoned the use of wooden flagpoles and is using steel flagpoles. For ordinary building, 60 ft in height above the curb, a pole 43½ ft in height is us which is sufficient for the tackle of a large or post-flag, for the reason that m parapets are very low. Each pole is required to be fitted complete with a ca iron, galvanized, revolving truck, mounted on crucible-steel pins, the cap b neath it, also, being of galvanized iron. The truck is fitted with two 4 bronze sheaves on Tobin-bronze pins, surmounted with an 8-in 20-oz copp ball, acid-cleaned and painted with four coats of the best English weather-pr sizing, and covered with XXXX leaf-gold. One or more field-joints are p mitted in the length of the pole, which are determined according to standa details, the bands being secured to the male tube, and both edges of the in band and the shoe being machine-beveled to insure a perfect fit. The fema tube is drilled and secured to the male shoe with tap-screws of sufficient streng to carry the upper section of the pole, and the ends of the screws are ups The exposed ends of the female tube are chamfered and caulked tight. A sa collar or band, to receive the copper flashing, is secured to the pole and bra just above the roof-lines.

Dimensions of Schoolrooms, Boston Schools.‡ The sizes of the rooms i the Boston school-houses, as adopted by the school board, are, for gramm schools, 28 by 32 ft in plan by 13 ft 6 in in height; for primary schools, 24 l 32 by 12 ft. This accommodates 56 scholars per room, in each grade, allowi 216 cu ft per scholar in the grammar schools, and 165 cu ft in the primary gra A width of 27 ft is very satisfactory for schoolrooms, and is commonly adopt because it permits of the use of 28-ft joists, without waste.

Heights of Blackboards in Schoolrooms.‡ The heights from floor to t of chalk-rail should be about as follows:

For third and fourth grades,	chalk-rail......	2 ft 1 in from floor
For fifth grade,	chalk-rail......	2 ft 2½ in from floor
For sixth grade,	chalk-rail......	2 ft 4 in from floor
For seventh and eighth grades,	chalk-rail......	2 ft 6 in from floor

Slate blackboards are made 3 ft 6 in, 4 ft and 4 ft 6 in high, 4 ft being a v common and satisfactory height.

* The Building Trades Pocketbook.

† From data compiled by E. S. Hand from notes furnished by C. B. J. Snyder, Sup intendent of School Buildings, New York City.

‡ F. E. Kidder, in previous editions.

Sizes of Seats and Desks for Schools and Academies *

Number of desk	Age of scholar (years)	Height of seat or chair (in)	Height of desk (next scholar) (in)	Space occupied by desk and seat (back to back) ft	in
0	16 to 18	16¾	29½	2	9
1	14 to 16	15½	28	2	9
2	12 to 14	15½	27½	2	8
3	10 to 12	14¼	26½	2	7
4	8 to 10	13¼	25½	2	5
5	7 to 8	12¼	24	2	4
6	6 to 7	11½	22½	2	3
7	5 to 6	10½	21	2	2
....	4 to 5	9½	19	2	0

Desks for two scholars are 3 ft 10 in long, and for a single scholar, 2 ft long.
Aisles are from 2 ft to 2 ft 4 in wide, according to age of scholars and size of room.

Additional Data† on School-Houses

Sizes of Rooms. The Department of Education, New York City, has adopted, the dimensions of the schoolrooms, the German standard of 22 by 30 ft in an by 14 ft in height, with unilateral lighting. These dimensions are used for grades of elementary schools, the sittings being on the basis of 15 sq ft of or-space per pupil. Good light cannot be had on desks which are placed at a eater distance from the windows than one and one-half times the height of the of the upper sash from the floor.

Sizes of Seats and Desks for Elementary and High Schools

Number of desk	Age of scholar (years)	Height ‡ of seat or chair (in)	Height ‡ of desk (in)	Space§ occupied by desk and seat (back to back) (in)
0	16 to 18	17	31	32
1	14 to 16	16	30	32
2	12 to 14	15	28	31
3	10 to 12	14	26	30
4	8 to 10	13	24	29½
5	7 to 8	12	23	27
6	6 to 7	11	22	27
7	5 to 6	10	20½	26

lackboards. For first and second-year scholars the chalk-rail is placed 2 ft n the floor, and the boards are 4 ft high. This allows the smaller children

F. E. Kidder, in previous editions.
From data compiled by E. S. Hand from notes furnished by C. B. J. Snyder, Superndent of School Buildings, New York City.
Heights are measured as follows: From the floor to the top of ink-well strips of s, and from floor to top of front edge of seats, and should not vary more than ½ in n the heights given in this table.
isles have a minimum width of 18 in for the lower grades and 22 in for the upper les.
If chairs are used, this distance must be increased from 1½ to 2 in.

Number of treads or risers

								6		7		8		9		10		11		12		13		14	
Story-height or horizontal length of run																									
ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in
1	0	1	6	2	0	2	6	3	0	3	6	4	0	4	6	5	0	5	6	6	0	6	6	7	0
1	0½	1	6¾	2	1	2	7¼	3	1½	3	7¾	4	2	4	8¼	5	2½	5	8¾	6	3	6	9¼	7	3½
1	1	1	7½	2	2	2	8½	3	3	3	9½	4	4	4	10½	5	5	5	11½	6	6	7	0½	7	7
1	1½	1	8¼	2	3	2	9¾	3	4½	3	11¼	4	6	5	0¾	5	7½	6	2¼	6	9	7	3¾	7	10½
1	2	1	9	2	4	2	11	3	6	4	1	4	8	5	3	5	10	6	5	7	0	7	7	8	2
1	2¼	1	9⅜	2	4½	2	11⅝	3	6¾	4	1⅞	4	9	5	4⅛	5	11¼	6	6⅜	7	1½	7	8⅝	8	3¾
1	2½	1	9¾	2	5	3	0¼	3	7½	4	2¾	4	10	5	5¼	6	0½	6	7¾	7	3	7	10¼	8	5½
1	2¾	1	10⅛	2	5½	3	0⅞	3	8¼	4	3⅝	4	11	5	6⅜	6	1¾	6	9⅛	7	4½	7	11⅞	8	7¼
1	3	1	10½	2	6	3	1½	3	9	4	4½	5	0	5	7½	6	3	6	10½	7	6	8	1½	8	9
1	3¼	1	10⅞	2	6½	3	2⅛	3	9¾	4	5⅜	5	1	5	8⅝	6	4¼	6	11⅞	7	7½	8	3⅛	8	10¾
1	3½	1	11¼	2	7	3	2¾	3	10½	4	6¼	5	2	5	9¾	6	5½	7	1¼	7	9	8	4¾	9	0½
1	3¾	1	11⅝	2	7½	3	3⅜	3	11¼	4	7⅛	5	3	5	10⅞	6	6¾	7	2⅝	7	10½	8	6⅜	9	2¼
1	4	2	0	2	8	3	4	4	0	4	8	5	4	6	0	6	8	7	4	8	0	8	8	9	4
1	4½	2	0¾	2	9	3	5¼	4	1½	4	9¾	5	6	6	2¼	6	10½	7	6¾	8	3	8	11¼	9	7½
1	5	2	1½	2	10	3	6½	4	3	4	11½	5	8	6	4½	7	1	7	9½	8	6	9	2½	9	11
1	6	2	3	3	0	3	9	4	6	5	3	6	0	6	9	7	6	8	3	9	0	9	9	10	6
1	7	2	4½	3	2	3	11½	4	9	5	6½	6	4	7	1½	7	11	8	8½	9	6	10	3½	11	1
1	8	2	6	3	4	4	2	5	0	5	10	6	8	7	6	8	4	9	2	10	0	10	10	11	8
1	9	2	7½	3	6	4	4½	5	3	6	1½	7	0	7	10½	8	9	9	7½	10	6	11	4½	12	3
1	10	2	9	3	8	4	7	5	6	6	5	7	4	8	3	9	2	10	1	11	0	11	11	12	10
2	2	3	3	4	4	5	5	6	6	7	7	8	8	9	9	10	10	11	11	13	0	14	1	15	2
2	4	3	6	4	8	5	10	7	0	8	2	9	4	10	6	11	8	12	10	14	0	15	2	16	4

... can be used for either treads or risers

Height of riser or width of tread, inches	Number of treads or risers																									
	15		16		17		18		19		20		21		22		23		24		25		26		27	
	Story-height or horizontal length of run																									
	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in	ft	in
6	7	6	8	0	8	6	9	0	9	6	10	0	10	6	11	0	11	6	12	0	12	6	13	0	13	6
6¼	7	9¾	8	4	8	10¼	9	4½	9	10¾	10	5	10	11¼	11	5½	11	11¾	12	6	13	0¼	13	6½	14	0¾
6½	8	1½	8	8	9	2½	9	9	10	3½	10	10	11	4½	11	11	12	5½	13	0	13	6½	14	1	14	7½
6¾	8	5¼	9	0	9	6¾	10	1½	10	8¼	11	3	11	9¾	12	4½	12	11¼	13	6	14	0¾	14	7½	15	2¼
7	8	9	9	4	9	11	10	6	11	1	11	8	12	3	12	10	13	5	14	0	14	7	15	2	15	9
7⅛	8	10⅞	9	6	10	1⅛	10	8¼	11	3⅜	11	10½	12	5⅝	13	0¾	13	7⅞	14	3	14	10⅛	15	5¼	16	0⅜
7¼	9	0¾	9	8	10	3¼	10	10½	11	5¾	12	1	12	8¼	13	3½	13	10¾	14	6	15	1¼	15	8½	16	3¾
7⅜	9	2⅜	9	10	10	5⅜	11	0¾	11	8⅛	12	3½	12	10⅞	13	6¼	14	1⅝	14	9	15	4⅜	15	11¾	16	7⅛
7½	9	4½	10	0	10	7½	11	3	11	10½	12	6	13	1½	13	9	14	4½	15	0	15	7½	16	3	16	10½
7⅝	9	6⅜	10	2	10	9⅝	11	5¼	12	0⅞	12	8½	13	4⅛	13	11¾	14	7⅜	15	3	15	10⅝	16	6¼	17	1⅞
7¾	9	8¼	10	4	10	11¾	11	7½	12	3¼	12	11	13	6¾	14	2½	14	10¼	15	6	16	1¾	16	9½	17	5¼
7⅞	9	10⅛	10	6	11	1⅞	11	9¾	12	5⅝	13	1½	13	9⅜	14	5¼	15	1⅛	15	9	16	4⅞	17	0¾	17	8⅝
8	10	0	10	8	11	4	12	0	12	8	13	4	14	0	14	8	15	4	16	0	16	8	17	4	18	0
8¼	10	3¾	11	0	11	8¼	12	4½	13	0¾	13	9	14	5¼	15	1½	15	9¾	16	6	17	2¼	17	10½	18	6¾
8½	10	7½	11	4	12	0½	12	9	13	5½	14	2	14	10½	15	7	16	3½	17	0	17	8½	18	5	19	1½
9	11	3	12	0	12	9	13	6	14	3	15	0	15	9	16	6	17	3	18	0	18	9	19	6	20	3
9½	11	10½	12	8	13	5½	14	3	15	0½	15	10	16	7½	17	5	18	2½	19	0	19	9½	20	7	21	4½
10	12	6	13	4	14	2	15	0	15	10	16	8	17	6	18	4	19	2	20	0	20	10	21	8	22	6
10½	13	1½	14	0	14	10½	15	9	16	7½	17	6	18	4½	19	3	20	1½	21	0	21	10½	22	9	23	7½
11	13	9	14	8	15	7	16	6	17	5	18	4	19	3	20	2	21	1	22	0	22	11	23	10	24	9
13	16	3	17	4	18	5	19	6	20	7	21	8	22	9	23	10	24	11	26	0	27	1	28	2	29	3
14	17	6	18	8	19	10	21	0	22	2	23	4	24	6	25	8	26	10	28	0	29	2	30	4	31	6

* The editor is indebted to T. Z. Talley for the calculations and arrangement of this table.

to use the lower portion. The upper part of the surface is at a height convenient for the use of the teacher, there being much display-work employed in the lower grades. For scholars in grades from the third to the eighth year, inclusive, and for high schools the chalk-rail is placed 2½ ft from the floor and the boards are 3 ft 6 in in height.

Doors and Stairways. Wardrobes should be entered from the classroom only. Classroom-doors should open into the rooms, so as to afford the teacher control in case of panic. All exit-doors should open out. All stairways should be shut off from corridors by means of self-closing doors, which, together with the stairways and the enclosures, should be of fire-proof materials. Stairways should be of sufficient number to permit of the building being vacated within three minutes from the time a signal is given. This can be effected by allowing a linear width of 4 ft for the first 50 persons and 12 in additional for each 50 persons in excess thereof. No stairway is to be less than 4 ft nor more than 5 ft in width. Exits should be planned so as to provide 15 lin ft for the first 50 persons and 6 in additional for each 100 persons in excess thereof. No stairway should have more than 15 steps in any one flight, changes in direction being effected by a square platform and no winders being used. No stair-door or exit-door should open out over a step. Platforms are to be provided for such doors and are to extend at least 1 ft beyond the edge of the door when standing open.

Stairs.* The RISE of a stair is the height from the top of one step to the top of the next. The TOTAL RISE is the height from floor to floor. The RUN is the horizontal distance from the face of one riser to the face of the next. RISERS are the upright boards or other materials forming the faces of the steps, and the TREADS are the horizontal pieces or surfaces on which the feet tread. Treads are usually from 1¼ to 1¾ in wider than the run, on account of the NOSING. The height of an individual riser or the RISE of any stairs is found by dividing the TOTAL RISE by the number of risers. The RUN of the stairs may be fixed at will unless the space is cramped, but to secure a comfortable stair the run must bear a certain relation to the rise.

Rules for Dimensions of Treads and Risers. For ordinary use a rise of from 7 to 7½ in makes a very comfortable flight of stairs. For schools and for stairs used by children the rise should not exceed 6 in. Stairs having a rise greater than 7¾ in are steep. The width of the run should be determined by the height of the rise; the less the rise the greater should be the run, and *vice versa*. Several rules have been given for proportioning the run to the rise:

(1) THE SUM OF THE RISE AND RUN should be equal to from 17 to 17½ in.

(2) THE SUM OF TWO RISERS AND A TREAD should not be less than 24 nor more than 25 in.

(3) THE PRODUCT OF THE RISE AND RUN should not be less than 70 nor more than 75.

These rules apply only to stairs with nosings. Stone stairs without nosings should have at least 12-in treads for adults. (See Tables, pages 1646–7.)

Height of Hand-Rail. In dwellings, hotels, apartments, etc., the height of the rail should be about 2 ft 6 in above the tread, on a line with the face of the riser. For grand staircases the height may be reduced to 2 ft 4 in. On steep stairs the height should be from 2 ft 7 in to 2 ft 9 in. The rail should also be raised over winders. On landings, the height of the rail should be equal to the height of the stair-rail, measured at the center of the tread, the usual height in residences being from 2 ft 8 in to 2 ft 10 in.

* This subject is quite fully treated in Building Construction and Superintendence, Part II, Carpenters' Work, by F. E. Kidder.

Sash-Cords.* Until a few years ago, linen or cotton cord only was used for nnecting weights with the sashes of double-hung windows, and cord is still ore extensively used than either ribbons or chains. For windows of ordinary e a good brand of cord will wear for a long time, and this material will probly never be entirely displaced by metal. "Tests made at the Massachusetts stitute of Technology show that cords wear much longer than chains, though ey have less tensile strength. Cords should be smooth and round, so that each and bears its part of the stress, and well glazed, so that they have a smooth rface and consequently less wear from friction with the wheel of the pulley." has been found that cord can be braided too hard for durability, yet if it is aided so as to be very flexible it may be so soft that it will stretch and cause eat annoyance by permitting the weight to hit the bottom of the weight-box. e architect, however, should always specify the particular BRAND and SIZE cord to be used, and also the diameter of the pulley. Among the leading ands of sash-cord at present are the Samson Spot,† and the Silver Lake A.‡ ese brands are superior to the ordinary braided cords, which are made from erior yarns to meet the jobbers' requirements for price. In addition to other ost excellent qualities, the Samson cord offers an additional advantage that chitects will appreciate; it has a colored strand woven through it, which shows spots on the surface and thus enables one to tell at a glance that no other cord s been substituted. The Silver Lake A sash-cord has the name Silver Lake A anded on every foot of cord; but unless the letter A accompanies the name a cond grade of cord is denoted. The marking of the cord by color, or any ner device, does not alter the quality of the cord. Special marks may be plied to inferior cords as well as to the best. The following numbers should specified for the different weights of sash-weights:

Relative Sizes of Sash-Cords, Weights and Pulleys

ize-number	6	7	8	9	10	12
Diameter in inches	3/16	7/32	1/4	9/32	5/16	3/8
eet per pound	66	55	44	36	27	20
uitable for weights in pounds up to	5	12	20	30	40	50
inimum diameter in inches of pulley allowable	1 1/2	1 3/4	2	2 1/4	2 1/2	3

For hanging sashes weighing over 40 lb, only the largest size of Samson or ver Lake A cord, or some form of sash-chain or sash-ribbon, should be used, d the pulleys should be selected to fit the cord or chain. A guarantee that the d will last at least twenty years may be had from either of the manufacturers ntioned above. The Samson wire-center sash-cord has recently been put on market. This is really a metal sash-cord protected by a braided-cotton sure which acts as a noiseless cushion. It is claimed that it harmonizes with the dow-finish and that it has greater durability than other sash-cords or metal ices. (See record of tests made at Massachusetts Institute of Technology, ge 1651.) The standard color is that of dark mahogany, but this cord is de to order for large buildings in other colors to match the finish.

* The following notes, relating to Sash-Cords, Sash-Chains, Sash-Ribbons, Sashights and Sash-Balances, are condensed and revised from articles by Professor Thomas lan. in Kidder's Building Construction and Superintendence, Part II, Carpenters' rk.
† Manufactured by the Samson Cordage Works, Boston, Mass.
‡ Manufactured by the Silver Lake Company, Boston, Mass.

Sash-Chains. Of several styles of sash-chains on the market, the style most largely used is the flat-link chain.* This chain is made either of steel, or of bronze composed of 95% copper and 5% of tin. For suspending very heavy sashes, doors and gates, a cable-chain has been extensively used. Star † sash-chain is made of bronze-metal. The manufacturers of the Norris sash-pulley claim that a riveted chain that has joints only one way is almost sure to break when even slightly twisted, and that it is better to use two chains of the link-pattern running side by side over the same pulley. The strongest sash-chains are of steel made rust-proof by the hot-galvanizing process, and electro-copperplated to give a bronze finish; and of a bronze-mixture which looks like copper, but tougher and harder. One firm ‡ claims that its galvanized-steel sash-chain is from 11 to 45% stronger than any bronze or copper sash-chain and that it will resist fire for a much longer period. The tensile strength of their chain varies from 475 to 850 lb, according to the weight used.

Sash-Ribbons. These are now also extensively used in hanging the sashes in the better class of buildings. The ribbons are made of steel and aluminum bronze or of some mixture of aluminum, and in 3/8, 1/2, 5/8, 3/4 and 7/8-in widths. They are claimed to be practically indestructible, but according to one series of tests it would appear that in some cases they do not wear as long as sash-cords or sash-chains. Some people object that the ribbons snap against the pulley stiles, when the sash is raised or lowered, and thus make considerable noise. The 3/8-in ribbon may be used for a sash weighing up to 100 lb and requires 50-lb weights. For a window 6 ft 10 in high and 3 ft wide, glazed with plate glass, the ribbons with attachments cost about 75 cts. Sash-ribbons are now manufactured by a number of firms who also make the necessary attachments for weight and sash. For the best working of windows hung with ribbons, pulleys of the following sizes should be used:

For sashes weighing not over	40 lb,	2 in
For sashes weighing not over	60 lb,	2 1/4 in
For sashes weighing not over	100 lb,	2 1/2 in
For sashes weighing not over	150 lb,	3 in
For sashes weighing not over	250 lb,	3 1/2 in
For sashes weighing not over	300 lb,	4 in
For sashes weighing not over	350 lb,	4 1/2 in

Comparative Strength of Sash-Cords and Chains. The comparative strength and durability of sash-cords and chains have been determined by careful tests, but there is a great variation in both cases, due partly to variation in material, but principally to the relative sizes of the chain and pulley or cord and pulley. The cords or chains may be too light for the weights used, or the pulleys too small in diameter to carry the cord without undue bending. The pulleys may also have too narrow a groove or an uneven groove with sharp edges which cut the cords. The larger the diameter of the pulley, the less the wear.

Tests § on Wire-Center Sash-Cord and Bronze Sash-Chains. The cord tested was size No. 8, 1/4-in diam, Samson solid braided cotton cord with a

* One type of this kind of sash-chain is manufactured by the Bridgeport Chain Company, Bridgeport, Conn.

† Manufactured by the U. T. Hungerford Brass & Copper Company, New York City.

‡ The Oneida Community, Ltd., Oneida, N. Y.

§ Made at the Massachusetts Institute of Technology, May, 1914, by Professor Miller.

wire cable center, 1/16 in in diam. The chains tested were of two different makes of bronze, size No. 2, purchased in the open market as typical bronze sash-chains, each recommended by a reputable dealer as the proper chain for use with a 25-lb window-weight. The tests for the better of the two chains are those given. Durability-tests were made by raising and lowering a 25-lb weight over a 2-in pulley, each movement corresponding to once opening and shutting a window. The cord was tested over the regular round grooved pulley ordinarily used for cords, and the chains were tested over the combination grooved pulley usually furnished for sash-chains. For the fire-tests the cords or chains were hung through an asbestos box in which a Bunsen flame under pressure was applied to all alike, the temperature being about 2 200° F. A 25-lb weight was attached in each case to keep the cord or chain under the same tension. The wire-center cord took about twice as long to burn through and wore about seventeen times as long as the bronze chain.

Tests on Wire-Center Sash-Cord and Bronze Sash-Chain

Durability-tests		Fire-tests	
Number of lifts before breaking		Length of time before parting	
Bronze chain	Samson wire-center cord	Bronze chain, sec	Samson wire-center cord, sec
34 944	659 892	42.5	78.5
37 486	592 559	40	75.5
37 381	632 230	39	77
32 948	594 114	32	75
40 356	631 286	....	....
31 234	577 154	....	....
40 790	504 032	....	....
27 874	637 796	....	....
Average 35 377	Average 603 633	Average 38.4	Average 76.5

Weights of Sashes and Glass. In figuring the weights of windows, the weight of the glass may be taken at 3½ lb per sq ft for plate glass, 1½ lb for double-strength glass and 1 lb for single-strength glass. For the weight of the wooden sash, add together the height and width, in feet, of each sash, and multiply by 2.1 for 2¼-in sash, by 1⅔ for 1¾-in sash and by 1⅓ for 1⅜-in sash. These values are sufficiently accurate for determining the size of sash-cords and pulleys, but the weights should be determined by weighing each sash after it is glazed, as the weight of the glass varies considerably.

Iron Sash-Weights. The weights ordinarily used for balancing windows are made of cast iron, in the form of solid cylinders from 1⅜ to 2⅛ in in diameter, and from 7½ to 31 in long, with an eye cast in the upper end of each. The lengths vary with the weights, which are from 2 to 25 lb. Flat weights, which usually are called for in the Philadelphia and some other markets, are from 6 to 34½ in long, from 2 to 30 lb in weight, and from 1¼ by 1⅝ to 1⅝ by 2¼ in in cross-section. In ordering sash-weights the number of pounds of each weight, and the sections and lengths of the boxes in which the weights will work, should be given. Ordinary weights have very rough eyes for the sash-cords. There are a few manufacturers in the East who make weights with a patent eye that will

not cut the cord. A sectional sash-weight* made with a well-designed hooking device which has given satisfaction, is said to be one of the best on the market. Usually from three to six sections are required on each side to balance a sash properly. If the hooking-device fails near the top the upper sash cannot be closed and if at the bottom the window cannot be opened. It is then necessary to open the weight-box and rehang the sections before the window can be operated. In theory, sectional weights are ideal; in practice, however, they are not considered as satisfactory as solid weights. The Brown † sectional weights are made 2¼ by 2⅝ in and in weights of 6, 7, 8, 9 and 10 lb. They are made of both cast-iron and lead. It frequently occurs after a contract is let, that the glass is changed from double-thick to plate or prism glass. This means increased weight; but the length of the sash-weight cannot be increased and it, therefore, becomes necessary to increase the area of its cross-section. If the weight-box is detailed to take the regular round sash-weight, its general construction will be such that it will take a 2-in round sash-weight, but not a 2-in square sash-weight. This difficulty can be avoided by a little thought at the start. An added depth of ¼ in in the weight-box permits the use of a rectangular cast-iron sash-weight. The Sanborn sectional sash-weight ‡ is intended for use in large buildings of heavy construction. Because of the lack of uniformity in the weight of plate glass the required weights of sash-weights cannot be accurately determined previous to the hanging of the sashes. By the use of a sectional sash-weight, combinations of units can be made up to suit the requirements. The units are made square or rectangular in section in order to secure a maximum weight with a minimum length. An opening of 12 in in the side of the pocket is sufficient for hanging the largest unit. These units are manufactured in standard sizes to meet the general conditions found in the building trades.

Lead Sash-Weights. It often happens that for wide and low windows the weights if of iron would be so long that they would touch the bottom of the pocket before the bottom sash was fully raised. In such cases lead weights are usually resorted to, lead being 80% heavier than cast iron. By casting the weights square in section, whether of iron or lead, a considerable saving can be made in the lengths. One sash-weight manufacturer § makes a specialty of compressed-lead sash-weights, with wrought and malleable-iron fastenings centered so that the weights hang perfectly plumb; and when lead weights are necessary the architect will do well to specify the weights made by this company. These weights are made under hydraulic pressure, by which greater smoothness, solidity and density of metal is secured than is possible by the casting-process. A wrought-iron rod is run through the center, to which are securely attached the malleable-iron fittings. In hanging the sashes the weights for the upper sash should be about ½ lb heavier than the sash, and for the lower sash, ½ lb lighter.

Sash-Balances. Within a comparatively few years several devices have been patented for balancing sashes by means of springs instead of weights, but the author believes that only one type, known as the SASH-BALANCE, has proved a practical success. The sash-balance consists of a drum on which the ribbon is wound, and which contains a coiled-steel clock-spring, immersed in oil; the spring sustains the weight of the sash. The common type very much resembles in outward appearance the ordinary sash-pulley, and is applied in practically the same way; the ribbons, which are made usually of aluminum-bronze, are

* Manufactured by E. E. Brown & Company, Philadelphia, Pa.
† Manufactured by E. E. Brown & Company, Philadelphia, Pa.
‡ Manufactured by the Lidgerwood Manufacturing Company, New York City.
§ Raymond Lead Works, Chicago, Ill.

ached to the sashes in the same manner as cords when weights are used. ile the sash-balance in its best form works very satisfactorily, it will probably er entirely supplant the weight and axle-pulley for ordinary windows. There many windows, however, for which sufficient pocket-room for weights cannot obtained without spoiling the effect desired or narrowing the glass, as in some windows, or where it is undesirable to break the frame into the brick jamb. such cases the sash-balance is almost invaluable. For hanging the glass rs of show-cases, sash-balances are usually preferable to weights. Sash-ances are made in both side and top-patterns, but the former are recommended erever there is room at the side of the frame for the depth of mortise required. windows of the sizes usually found in residences, the depth of the sash-ance measured from the face of the pulley-stile will vary from 3 to 4 in; s can be provided for usually by cutting a hole, if necessary, in the masonry studding back of the frame. As sash-balances require only a plank frame, consequent reduction in the cost of the frame offsets the extra cost of the ance. In remodeling old buildings which have plank frames without weights, h-balances are found to be a great convenience, since they can easily be in-ted in the old frames. An advantage which all spring-balances possess is t they act most strongly when the sash is down, and so enable one to raise inding window more readily than if it were hung with weights; while when sash is up the springs barely suffice to hold it in position, and do not offer stance to drawing it down. Of the various sash-balances on the market, the lman * and the Caldwell † are the most extensively used, and are undoubtedly able. The Pullman Unit sash-balance has been on the market many years l has proved satisfactory. These balances are now made with uniform-size e-plates for the various weights of sash with which they are to be used, and s make it possible to have all mortises for the balances cut at the mill, as is v done for the regular cord-pulleys. The Caldwell sash-balance, both top l side-types, is much used by the United States Post Office and Navy Depart-nts. It is used also by the leading car-builders. The springs are made of h-grade cold-rolled tempered-steel wire, a material similar to that used for k-springs. The manufacturers guarantee these sash-balances for from ten ifteen years.

Seating-Space in Churches and Theaters. The minimum spacing for rs, back to back, is 30 in. This spacing is fairly comfortable for occupants, is a little cramped for persons passing by others into or out of the pews. pacing of 32 in is to be preferred, and if there is abundance of room, the spac-may be made 33 in. Anything over 33 in is a waste of room. A space of in in the length of the pew is considered a SITTING.‡

Opera or Theater-Chairs are made 19, 20, 21 and 22 in wide, center to center arms, and in arranging them in rows where the aisles converge, the ends are ught to a line on the aisles by using a few chairs that are either narrower or ler than the standard width. For churches, a standard width of 20 in is least that is desirable. For theaters, 21 or 22-in chairs are commonly used the parquet, 20 or 21-in in the dress-circle, and 20 and 19-in in balcony and lery, although there is no accepted rule in this respect. On account of the t-lifting, opera or theater-chairs may be comfortably spaced 31 in, back to k, and this is the usual spacing in halls and churches. In theaters the irs are usually set on steps. In the upper gallery these steps should not be re than 30 in wide; in the balcony they are usually made either 30 or 31 in

* Manufactured by the Pullman Manufacturing Company, Rochester, N. Y.
† Manufactured by the Caldwell Manufacturing Company, Rochester, N. Y.
‡ For dimensions of pew-bodies see page 48 of Churches and Chapels, by F. E. Kidder.

wide, and in the parquet, 31 or 32 in wide. As a rule the higher-priced seats are more commodious than the lower-priced.

Estimating Seating Capacity. The actual seating capacity of theaters and audience-rooms can be determined only by drawing the seats to an accurate scale, on the floor-plan, and then counting the number of chairs, or measuring the linear feet of pews.

Approximate Seating Capacity. For approximate purposes the seating capacity or required size of room may be determined by allowing from 7 to 8 sq ft to each seat, or sitting, when on a curve, and from 6 to 7 sq ft to each sitting when in straight rows, the smaller number being used only for large rooms. This allows for aisles and pulpit-platform. For small concert-halls and narrow rectangular rooms, 6 sq ft per sitting will usually be sufficient allowance, provided only that the actual floor-space utilized for seats and aisles is considered.

Seating Capacity of Several of the Older Cathedrals, Churches, Theaters and Opera-Houses *

European Cathedrals and Churches

Estimating that one person occupies an area of 19.7 inches square †

St. Peter's, Rome	54 000	Notre Dame, Paris	21 000
Milan Cathedral	37 000	Pisa Cathedral	13 000
St. Paul's, Rome	32 000	St. Stephen's, Vienna	12 000
St. Paul's, London	25 600	St. Dominic's, Bologna	12 000
St. Petronio's, Bologna	24 400	St. Peter's, Bologna	11 [illegible]
Florence Cathedral	24 300	Cathedral of Sienna	11 000
Antwerp Cathedral	24 000	St. Mark's, Venice	7 000
St. Sophia's, Constantinople	23 000	Spurgeon's Tabernacle, London	7 000
St. John Lateran, Rome	22 900		

European Theaters and Opera-Houses

Carlo Felice, Genoa	2 560	Drury Lane, London	1 9[illegible]
Opera-House, Munich	2 370	Covent Garden, London	3 0[illegible]
Alexander, St. Petersburg	2 332	Opera House, Berlin	1 6[illegible]
San Carlo, Naples	2 240	Adelphi, London	2 [illegible]
Imperial, St. Petersburg	2 160	Lancaster, London	1 [illegible]
La Scala, Milan	2 113	Globe, London	1 1[illegible]
Academy of Paris	2 092	....	......

Some Early American Theaters and Opera-Houses, outside of New York

The Auditorium, Chicago	4 200	Castle Square Theater, Boston	1 600 to 1 800
Academy of Music, Philadelphia	3 124	Gaiety Theater, Boston	nearly 3 000
Boston Theater, Boston	3 000	Grand Opera-House, Cincinnati	1 [illegible]
....	....		

* The table following this one gives the seating capacities of theaters in use in 1911 in some of the boroughs of New York City. The above table of seating capacities of some of the earlier churches and theaters is retained for purposes of comparison. So many important structures of these types have been erected in recent years in the larger cities of the world, or are now in process of erection, that it has been found impossible to make any list that would be and would remain, for any length of time, complete.

† See note on page 1655.

Seating Capacity of New York Theaters (1914)

Boroughs of Manhattan and the Bronx			
Name	Seating capacity	Name	Seating capacity
ademy of Music	2 653	Gaiety	806
hambra	1 389	Garden	1 090
nerican	1 683	Garrick	844
nerican, Roof	1 134	Globe	1 194
tor	1 137	Gotham	1 626
lasco	984	Grand	1 888
rkley Lyceum	416	Grand Opera-House	2 093
jou	814	Greeley Square (Loew's)	1 906
oadway	1 776	Harlem Casino	*
onx	1 764	Harlem Opera-House	1 393
rnegie Hall	2 632	Harris	847
rnegie Lyceum	640	Herald Square	1 160
sino	1 465	Hippodrome	5200
ntury	2 078	Hudson	1 077
ntury, Roof	1 150	Hurtig and Seamon's (Music-Hall)	1 093
rcle	1 684	Illington	*
ty	2 289	Irving Place	1 079
inton Street (Odeon)	904	Keith's Union Square	1 080
orge M. Cohan	1 072	Kessler's 2nd Avenue	1 803
lonial	1 541	Kessler's 2nd Avenue, Roof	734
lumbia	1 315	Knickerbocker	1 351
medy	696	Lafayette	1 042
iterion	916	Liberty	1 211
aly's	1 074	Lincoln Square	1 547
elancy Street	1 242	Lipzin	1 030
xes (Dewey)	1 310	Little	299
th Street	1 436	Longacre	*
tinge	895	Loew's Fifth Avenue	964
npire	1 099	Loew's 7th Avenue	1 626
mily	687	Loew's National	2 333
fth Avenue (Proctor's)	1 204	Lyceum	957
th Street	1 255	Lyric	1 452
th Street (Brady's)	969	Madison Square Garden	3 366
lton	662		

* Data not furnished.

ote Regarding Unit Area for Seating Capacity. The unit area given in the e on page 1654 appears in the former editions of this book and seems to be small. The original authority for the data cannot be determined. A more onable minimum area would be about 23½ inches square, or about 18 by 3c or 540 sq in, or about 3.8 sq ft. Editor.

Seating Capacity of New York Theaters (1914) (Continued)

Boroughs of Manhattan and the Bronx

Name	Seating capacity	Name	Seating capacity
Madison Sq Garden, Roof	700	Proctor's 23rd Street	[illegible]
Manhattan Opera-House	3 200	Proctor's 58th Street	[illegible]
Maxine Elliott	904	Proctor's 125th Street	[illegible]
Metropolis	1 150	Prospect	[illegible]
Metropolitan Opera-House	3 305	Republic	[illegible]
McKinley Square	1 500	Richmond	[illegible]
Miner's Bowery	1 168	Riverside	[illegible]
Miner's Bronx (Acme)	1 798	Savoy	[illegible]
Miner's 8th Avenue	1 178	Star	[illegible]
Minsky	1 866	St. Nicholas	*
Moulin Rouge	1 615	Thalia	[illegible]
Murray Hill	1 224	Third Avenue (Keeney's)	[illegible]
Mount Morris	*	39th Street	[illegible]
Nemo	941	Tremont	[illegible]
New Amsterdam	1 618	Victoria	[illegible]
New Amsterdam, Roof	610	Victoria, Roof	[illegible]
New York Theater, Roof	1 337	Wadsworth	[illegible]
Olympic	745	Wallack's	[illegible]
116th Street	1 743	Washington	[illegible]
Odeon 145th Street	904	Weber's	[illegible]
Park	1 513	West End	[illegible]
People's	1 693	Weber and Field Music-Hall	[illegible]
Philipps	*	Winter Garden	[illegible]
Plaza	1 544	Yorkville	[illegible]
Playhouse	879		

* Data not furnished.

Borough of Brooklyn

Name	Seating capacity	Name	Seating capacity
Academy of Music	2 200	Jones	[illegible]
Amphion	1 589	Liberty	[illegible]
Bijou	1 562	Linden	[illegible]
Broadway	1 969	Lyceum	[illegible]
Bushwick	2 228	Lyric	[illegible]
Casino	1 503	Majestic	[illegible]
Columbia	1 673	Myrtle	[illegible]
Comedy	1 123	New Montauk	[illegible]
Crescent	1 610	Novelty	[illegible]
DeKalb	2 232	Olympic	[illegible]
Empire	1 740	Orpheum	[illegible]
Fifth Avenue	1 063	Oxford	[illegible]
Folly	1 840	Payton's	[illegible]
Fulton	1 492	Prospect Hall	[illegible]
Gayety	1 455	Royal	[illegible]
Gotham	958	Shubert's	[illegible]
Grand Opera-House	1 515	Star	[illegible]
Greenpoint	1 776		

Dimensions of Some Theaters and Opera-Houses **

ıe following are the dimensions, in feet, of some of the earlier theaters in this country in Europe.

Name and location	Auditorium			Proscenium-opening		Stage		
	Width	Depth	Height	Width	Height	Width	Depth*	Height‡
xander, St. Petersburg..	58	76	58	56	....	75	84	....
–, Berlin................	51	78	47	41	....	92	76	....
Scala, Milan............	71	95	64	49	....	86	78	....
Carlo, Naples..........	74	73	83	52	....	66	74	....
and, Bordeaux...........	47	56	57	37	....	80	69	....
le Lepeletier, Paris	66	76	66	43	....	78	82	....
vent Garden, London...	51	66	....	32	....	86	55	....
ury Lane, London.......	56	64	....	32	....	48	80	....
ston, Boston............	....	71	58	46	....	87	68	....
demy of Music, New York......................	62	87	....	48	....	83	71	....
demy of Music, Philadelphia..................	66	78	74	48	....	90	72	....
be, Boston.............	60	65	....	30	....	62	38	....
seum, Boston...........	68	61	....	31	....	68	46	....
ropolitan Opera-House, New York §............	....	....	....	54	50	100	73	88
Auditorium, Chicago..	....	....	....	....	....	110	70	95
pire, New York.........	69	66	....	34	34	67	30	65
ickerbocker, New York.	70¾	79	47⅔†	35	34	40	65½	....
rick, New York........	56⅙	52	....	27	....	71½	28⅛	....
h Avenue, New York...	....	....	....	....	....	80	35	....
erican, New York......	74½	74½	....	39	39	77¾	43⅓	73½
ctor's Pleasure Palace, New York..............	74⅓	74½	....	34	34‖	70	40	70
dson, New York........	67⅓	67	....	32	30	67⅓	30⅔	....
nd Opera-House, Cincinnati..................	67	69	....	36	34	67	41	....
tle Square, Boston¶.....	79½	85½	70†	40	34	68	45½	....
ety, Boston............	77	80⅔	....	....	....	60	42	70

tes on Theater-Dimensions.†† "The utmost distance from the front of age to the rear ought not to exceed 75 ft, or the limit the voice is capable panding in a lateral direction."

easured from the curtain-line, the San Carlo Theater in Naples is 73 ft; the

From the curtain or back line of proscenium opening.
Measured from stage to center of ceiling.
To the "gridiron" or rigging-loft.
As remodeled in 1893.
Can be enlarged to 40 by 40 ft.
The plan of this theater is in the shape of a horseshoe.
See footnote with table of Seating Capacities of Churches, Theaters, etc., page 1654, ard to data relating to recently constructed buildings of these types.
From The Planning and Construction of American Theaters, by Wm. H. Birkmire.

theater at Bologna, 74 ft. Of the London theaters, the Adelphi is 74 ft, Co
Garden 80 ft, the Gaiety 53 ft 6 in, Lancaster 58 ft 4 in, Marylebone 74 ft
the Globe 47 ft 6 in."

The width of the ideal theater, between inside walls, should be from
75 ft, and "the ceiling should be from 55 to 65, or even 70 ft above the
level."

"The depth of the parquet-floor at the orchestra-rail is governed by t
stage-level, and is generally from 3 ft 6 in to 4 ft 3 in below the stage. A d
of 3 ft 9 in is a good height, as it fixes the eye of the spectator 5 in above
stage-level."

"The height of the stage, that is, from the floor to the bottom of the 'gridi
or rigging-loft, should be 2 or 3 ft over twice the height of proscenium-openi
in order that the fire-curtain may be raised the full height of the openi
There should be a height of 7 ft above the gridiron to enable the fly-men to
their ropes with facility.

Proportioning Gutters and Conductors to the Roof-Surface. The
of gutters and down-spouts and their distance apart for roofs of mill-buildi
with a ¼ pitch and of different spans are shown in the following table: *

One-half roof-span, in feet.......	10	20	30	40	50	60	70	[illegible]
Size of gutter, in inches..........	5	5	6	6	7	7	8	[illegible]
Size of down-spouts, in inches...	3	3	4	4	5	5	6	[illegible]
Spacing of down-spouts, in feet..	50	50	50	50	40	40	50	[illegible]

The specifications of the American Bridge Company provide as follows
the size of gutters and conductors: †

Span of roof	Gutters	Conductors
Up to 50 ft	6 in	4 in every 40 ft
From 50 to 70 ft	7 in	5 in every 40 ft
From 70 to 100 ft	8 in	5 in every 40 ft

Hanging gutters should have a slope of about 1 in in every 16 ft.

"The Produce Exchange Building in New York City, with a roof-area
three-fourths of an acre, roughly speaking, has twelve leaders, each about
in diameter. The roof, which is paved with fire-brick, is graded with slopes
perhaps one in fifty toward the points at which the leader-openings are pl
most of these draining-surfaces being about 40 by 70 ft each. The provi
here made is equivalent to about 1 sq in of leader-opening to 140 sq ft of
surface. On the Sloane Building, at 19th Street and Broadway, New Y
City, with a roof-area of 18 000 or 20 000 sq ft, sloping one in twenty-five, th
are two leaders, each about 6 in in diameter, and a third rectangular lea
4 by 6 in in cross-section. This gives an allowance of 240 sq ft of surface to
square inch of leader-opening, while on the Massachusetts Hospital Life In
ance Company's Building and the Hemenway Building, in Boston, the proport
is only from 60 to 70 sq ft to the square inch of opening." ‡

* H. G. Tyrrell.
† M. S. Ketchum.
‡ Dwight Potter in The Technology Quarterly.

ELEVATOR-SERVICE IN BUILDINGS*

General Considerations. An efficient elevator-service may be obtained by achines of any one of several types, the choice of the one decided upon for any ilding depending upon varying conditions. The following is a general classi-ation of elevators (see, also, page 1668):

1. Hydraulic elevators:
   (1) Vertical, geared hydraulic type.
   (2) Horizontal, geared hydraulic type.
   (3) Direct-lift plunger-type.
   (4) Inverted (high pressure) plunger-type.
2. Electric elevators:
   (1) Drum-type.
   (2) Worm-gear traction-type.
   (3) Helical-gear type.
   (4) Gearless, traction-type.
      (*a*) Direct-drive (one-to-one) type.
      (*b*) Two-to-one type.

In addition to these, there are also the BELT-DRIVEN type of elevators, and the ND-POWER elevators. The belt-driven type may be either SINGLE-BELT or UBLE-BELT driven, the former being used with a reversible motor and the ter where driving-power is taken from a line-shaft. In view of varying and metimes conflicting claims of competing manufacturers, the architect's de-ion must be governed by impartial engineering judgment rendered after a reful study of the problem in each case.

Geared Versus Gearless Types of Elevators. (See, also, page 1669.) ere has been much discussion regarding the merits and demerits of geared d gearless machines for elevators and the efficiency and future of each. anufacturers of gearless traction-machines have claimed that the use of the lical gear, for example, for elevator-machines, being a relatively recent devel-ment, has not extended over a sufficient length of time to permit of extensive definite data; that they are used for different and less severe service than at for which the gearless traction-machines are employed; and that they can-t rival the gearless traction-machines from the standpoint of efficiency. On e other hand, the manufacturers of helical-gear elevator-machines claim that ars have been in successful use for many years, the substitution of helical ars for worm-gears being the only difference made in the application to their pe of elevators; that the helical-gear elevators installed in some of the highest ice-buildings are doing as much work as any gearless traction-machines; and at the mechanical efficiency of the helical-gear machines is only a little below at of the gearless traction, the electrical efficiency under local or ordinary nning conditions, greater, and the car-mile consumption in kilowatt-hours, less.

Questions of Cost and Efficiency of Elevators. The principal demerit elevator-machines of the gearless type is their relatively high first cost, al-ough even that is much lower than the initial cost of elevators of the plunger-pe. The use of any gear, whether of the helical, worm or spur-type, is, in the inion of many engineers, to be recommended only for the purpose of obtaining

* The matter relating to elevators and elevator-service is condensed and adapted by rmission, from data furnished by various engineers and manufacturers, and papers the Otis Elevator Company, New York City; The H. J. Reedy Company, Cincinnati, io; R. P. Bolton of The R. P. Bolton Company, Consulting Engineers, New York, d author of "Elevator Service"; C. E. Knox, Consulting Engineer, New York; M. W. rlich, Consulting Engineer, New York; and others.

a higher-speed motor, because a higher-speed motor costs less than a low-sp motor. The helical gear is generally considered a more efficient type of g than the worm-gear and has a deserved place in the development of the ele tor-industry. The helical-geared traction-elevators will undoubtedly be tensively used, for the reason that, even if they are not considered by s engineers to be as good in some details as machines of the gearless, tra type, they are less expensive. It is undoubtedly true, however, that the in duction of any gear means some loss in power, and it is claimed that tests that low-speed motors can be designed which are in every way as efficie high-speed motors. The data and statements in the following paragraphs re ing to elevator-service in buildings are presented as useful aids to architects include some opinions and conclusions which are to be accepted or modifie the light of constant additions to engineering knowledge.

A. General Conditions Affecting the Requirements of Specificatio for Elevator-Service *

Electric Versus Hydraulic Elevators. The question of the type of ele tors, whether electric or hydraulic, is best determined by the local conditions the special conditions which exist in every plant. The relation of the eleva equipment to the entire mechanical equipment should be carefully conside and should be decided only after mature deliberation and consultation w unprejudiced engineers and elevator-manufacturers. At the present time about 90% of the elevators being installed are electric, and this includes types of buildings from the small one with but one elevator to the tall s scrapers of the big cities. The electric equipment recommends itself, for it has all of the safety-features of the hydraulic equipment, it is a more fle system, is more adaptable to all kinds of conditions, and requires much less sp The question of space is a particularly important consideration in office-build Furthermore, the control-system is more automatic, the acceleration and reta tion of the car can be made more rapid, and the stops more accurate; the e ciency, also, is higher and in most cases the cost of operation lower. (See paragraph on Comparison of Merits of Electric, Traction and Hydraulic, Plu Elevators, page 1670.)

Location of Hoistways and Machinery-Room. The location of the hoi ways is rather a matter for the good judgment of the architect. In the la equipment all elevators serving one portion of the building and for the s character of service, should be placed in one BANK and not distributed or s rated. Thus, all express-elevators should be together and in one bank, as sho also, the locals. The hoistways should be so placed that the entrances, in stories, are on the same side of the car. In some of the larger cities, two e trances for a passenger-car are not permitted, unless the doors can be contr by the attendant without leaving the operating-device. The machinery-ro should be well ventilated, light and clean as possible, in order that the machin may be given proper attention. This room should also be large enough to m the machines readily accessible for repairs and inspection. Where the mach have heavy parts, which it may be necessary to remove from time to time repairs, it is advisable to locate a trolley-beam with hand-hoist above them facilitate the handling of these parts.

* The Otis Elevator Company, New York, has been of assistance in furnishing m of the engineering data of Section A, of this article on elevators. Among other it considers especially those conditions which should be considered and made de by the architect preliminary to the preparation of the elevator-specifications. paragraphs of Section A should be read in connection with those of Section B, page and the data compared.

lumber and Sizes of Elevators. (See, also, pages 1673 and 1675.) The
lber and sizes of elevators are governed by the following considerations: (1) character of the building, (2) the height of the building, (3) the rentable , (4) the time-intervals between the departures of cars, (5) the number of ies to be served, (6) the average number and length of stops per trip, (7) speed of the elevators and (8) the type of elevators used. No iron-clad s can be given for all types of buildings, but the larger office-buildings, buildings or light-manufacturing buildings have been sufficiently regular esign to warrant some general rules, based upon experience; even in these s, however, the governing conditions vary with the size of the building. of the most essential requirements for a satisfactory plant is QUICK SER- : and in first-class office-buildings the intervals between cars should not ed 30 seconds. The number of stories to be served by a car should be a ideration. For example, in a fifteen-story building, assuming that stops are e at 80% of the stories for one passenger each, and allowing 2 sq ft for each enger, and 4 sq ft for the operator, the car should have an inside area of it 28 sq ft. In order to facilitate unloading and thus increase the efficiency ie system, it is desirable to have the width of the car greater than the depth. he above case, a car with outside dimensions of about 6 ft wide by 5 ft deep ld give the best results, showing a difference of from 15 to 20% between the h and width. In specifying the equipment, it is better to call for several erate-sized cars and a high speed, than for a few large cars of slower speed larger capacities. Thus, three cars, each carrying one-third of all the passen- are better than two cars, each carrying one-half, as the service is increased naking the period between cars less. As the elevator-service largely deter- s the success of a building, it is of vital importance that a sufficient number evators be installed to handle the regular traffic, as well as emergency-con- ns in case of a shut-down. To illustrate what is considered the proper ortion of passenger-elevators for buildings of various heights, the following : is given, based upon a rentable area of 8 000 sq ft per story and 125 sq ft person. This table shows the various combinations for elevators with a d of from 400 to 500 ft and of 600 ft per min for buildings of from 10 to tories above the ground.

Table Showing Number of Elevators Required

mber tories	Express 600 ft per min	Local 600 ft per min	Express 600 ft per min	Local 500 ft per min	Express 500 ft per min	Local 400 ft per min
10	....	4	....	5	....	5
12	....	5	....	5	....	6
15	....	6	....	7	....	7
18	....	7	....	8	....	8
18	Express to 11 5	1 to 11 5	Express to 10 5	1 to 10 5	Express to 11 5	500 ft per min 1 to 11 5
20	....	All locals 8	....	....	....	....
20	Express to 12 5	1 to 12 5	Express to 11 5	1 to 11 5	Express to 12 6	1 to 12 6
25	Express to 15 6	1 to 15 6	Express to 14 6	1 to 14 6	Express to 15 7	1 to 15 7
[illegible]	Express to 18	1 to 18	Express to 17	1 to 17	Express to 18	1 to 18

Buildings equipped with both local and express-service should have the number of elevators for each class of service. In the case of the twenty story building for 600 ft-per-min speed, it is to be noted that the local elev are shown serving from the first to fifteenth story, whereas the express-elev serve from the fifteenth to the twenty-fifth story. The express-elevators not serve as many stories as the locals on account of the extra time consumed the run to the first express-landing. With the distribution as shown, the s for all stories is about equal, and both express-elevators and local elev operate on about the same schedule. In the fourth and fifth columns shown what is considered the best arrangement with the express-elev operating at a 600 and the locals at a 500 ft-per-min speed. Upon compari with the second and third columns, it will be noted that the express-elev are to serve one additional story. This is due to the difference in speed betw the express-elevators and local elevators and is done so that the schedule still remain the same for both. (See, also, paragraph on the Local and Exp Round-Trip Time, page 1675.)

Loads and Speeds. The sizes of the machines or hoisting-apparatus determined by the loads and speeds. The loads for passenger-cars should figured on a basis of 75 lb per sq ft of inside area of platform. The speed very important factor, as the foregoing indicates. This is usually limited local ordinances, and in New York City, cars stopping at all stories are not mitted to exceed a speed of 500 ft per min. For express-service, in that port of the shaft where no stop is made, a speed of 700 ft per min is allowed. Th NO-STOP DISTANCE must be at least 80 ft or more. The best companies for vator-insurance will not permit electric-drum elevators for a speed much o 400 ft per min, whereas the gearless, traction-drive type and the hydraulic typ are approved up to the limits, as noted above. In hydraulic plants it is necess to specify the number of round trips per hour for the entire elevator-equipment This is required in order to determine an adequate pumping-plant.

Hoistways. The hoistways should be finished to plumb-line dimensions that the car running on guide-rails set to plumb-line will at all points have same clearance. Supports should be provided adjacent to the hoistway for overhead beams at a distance, if possible, of at least 4 ft from the top of frame when the platform is flush with the top landing. This distance sho be increased where possible so that the car will have ample clearance, th preventing accidents due to striking the overhead work, in case it should past the top landing. The minimum clearances between the top of the c frame and the overhead apparatus are usually limited by the local building regu lations, and these should be consulted. In the case of the elevators operati at a speed greater than 350 ft per min, the distance given above would probab have to be increased in order to comply with these regulations. A pit should provided at the bottom of the shaft. This should be at least 3 ft deep, and is the case with the overhead clearances, the depth is usually regulated by t building regulations, in accordance with the speed of the elevator. Where possible, the hoistways should be so planned that the main guide-rails may b placed at the sides of the car. Supports should be provided at all the floors these rails, and where the distance between floors is greater than 12 ft, inte mediate supports should be provided. The distance from the supports for th overhead beams, to the penthouse or skylight-roof, varies with the type of in stallation, but can be accurately obtained from the elevator-manufacturer.

Protection of Counterweights. In New York City the Bureau of Buildi requires that where the counterweights run in the same shaft as the car, th must be protected with a substantial screen of iron from the top of the rail t

it 15 ft below, except where the plunger-type or traction-type elevator is l.

uilding Laws Governing Elevator-Installation. The Bureau of Build-, Borough of Manhattan, New York City, issued regulations * governing construction, inspection and operation of all types of elevators, and the ial attention of all architects is called to them, as they are not only obligatory e, but are excellent guides to practice at all times. The foregoing paragraphs intended to give an idea of what the architect must consider and provide building for the reception of the elevator-apparatus, and what he must rmine in order to enable the manufacturer to intelligently design and lay his machinery.

tandard Designs and Special Apparatus. The specifying of apparatus pecial construction is, as a rule, not to be recommended. Standard designs ıld be used as much as possible, as (1) they are more apt to be well designed, ed and built, (2) they are undoubtedly less expensive, both in initial cost maintenance and (3) repair-parts may be more easily and quickly obtained at less cost.

pecifications for Elevator-Installation. The specifications should include ı included in the following classification.

(1) Kinds of service and number of elevators of each service.
(2) Maximum load wanted.
(3) Maximum speed.
(4) Load with maximum speed.
(5) Maximum number of round trips per hour for each elevator.
(6) Method of control. For electric elevators, car-switch control should be used for passenger-service and for all elevators for a speed over 150 ft per min.
(7) Size of hoistways and area of car-platforms.
(8) Travel of car-platform in feet, number of car-landings, and number of openings at each landing.
(9) System used. If electric, direct or alternating current, the voltage and, also, the phase of cycles for alternating current should be given. If hydraulic, the steam-pressure or electric current characteristics for the pump-motors or the water-pressure, if the purchaser provides the pumps, tanks or other source of water-pressure supply.
o) A sketch-elevation showing landings, supports for overhead beams, space for the overhead sheaves, and runbys at top and bottom; a sketch-plan showing size and shape of hoistways, entrances, position of car and counterweight, guide-rails, and location of space available for machines, pumps, tanks, etc., with reference to hoistways.
1) Car and counterweight guide-rails, whether of wood or steel.
2) Supports for fastening the rails, character of these supports, and where and how located.
3) Value of finished car or cage, that is, the specified amount to be allowed for each, the design being subject to the approval of the architect.
4) Number and size of ropes, if not left to the judgment of the elevator-contractor. The largest sheaves possible should always be required, as this factor determines largely the life of the ropes.
5) System of signals, that is, (*a*) annunciators in the cars with push-buttons at the landings, (*b*) UP and DOWN signals in the cars, with UP and DOWN buttons at the landings, so arranged that a car going up receives only

* Published in the Record and Guide, July 29th, 1911.

UP signals, and a car going down receives only DOWN signals, the signal being automatically reset by the first car stopping at the floor from which the signal is given. This system adds greatly to the efficiency of a battery of elevators, as it avoids the confusion of more than one car answering a signal, or a car going in one direction stopping for a passenger going in the opposite direction. The number of stories at which each car is to land should always be specified.

(16) Indicators. Whether at the ground-floor only, for the information of the starter regarding the position of the cars, or at all floors. Indicators are unnecessary with the automatic signals last described, except at the ground-floor, as there is at each floor an UP and DOWN signal to show the first available car in either direction.

(17) Source of power. It should be specified whether the connections will be brought to the elevator-apparatus by the purchaser or by the elevator-contractor. If by the latter, a sketch should be made showing the distance, and for the electric system the specifications should state whether the wiring is to be open, that is, on cleats, in moldings, or in conduits; the sizes of wire, and the switches, cut-outs, etc. For the hydraulic system, the size of pipe for steam-supply should be given. The sizes of water-piping should be left to the elevator-contractor and he should be held responsible for them. Also, in the case of an hydraulic system operating from street-mains, the specifications should state by whom the piping is to be done and who is to furnish the water-meter.

(18) Pumps and tanks in hydraulic plants. These should be furnished by the contractor. The specifications should state whether the capacity is to be just ample to do the work, or whether there is to be a reserve capacity, with reserve-units, to provide against interference with the service in case of accident to a pump or tank, or for future elevators, but the sizes and design should be left to the judgment of a responsible elevator-contractor.

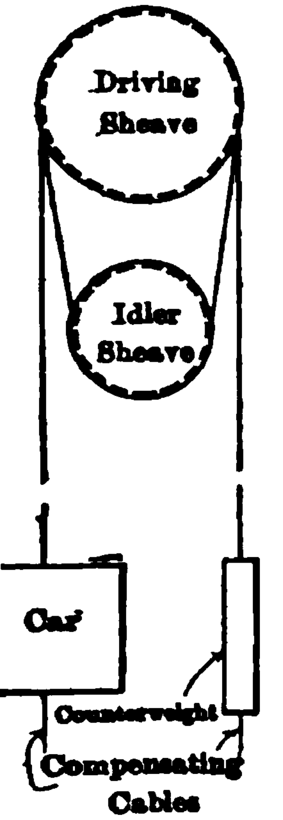
(19) Foundations for the machine, whether they are to be provided by the purchaser or by the contractor.

(20) Miscellaneous. Gratings underneath the overhead work, pitpans, painting in addition to the standard factory-finish and all items not mentioned above are generally furnished by the purchaser under separate contracts, but this should be clearly set forth in the elevator-specifications.

Safety-Devices for Elevators. (See, also, page 1672.) The question of safety-devices cannot be too carefully considered for all elevators, and for passenger-elevators in particular only the best and most thoroughly tested apparatus should be installed. Each car should be equipped with the mechanical device designed to grip the rails and stop the car in case it exceeds a predetermined maximum descending-speed, either from breaking of the cable or from any other causes. This safety-device should be mounted upon the car-frame beneath the platform, and should be operated by means of a speed-governor located overhead. For speeds above 150 ft per min, this gripping of the rails should be done gradually. In New York City the instantaneous stopping is not allowed above a speed of 100 ft per min. A switch for emergency use should be placed in the car of electric elevators. The opening of this switch should stop the car immediately and independently of the regular operating-device. All electric elevator-machines should be equipped with an electric brake. This brake should be automatically applied when the car stops or which

current-supply is interrupted. The brake should be released electrically and lied by means of spring-pressure. Automatic limits should be placed at top and bottom of the hoistway, to automatically slow down and stop the at the limits of travel, independently of the operator.

earless Traction-Elevators.* Among the more recent developments of elevator industry is the electric, gearless, traction-elevator (Figs. 1 and 2). e, also, Fig. 5.) The designing of an efficient slow-ed motor made it practical to build a traction-machine h the driving-sheave mounted directly upon the armature-shaft, thus eliminating the use of gears to reduce to the desired car-speed. This gearless machine is used for speeds from 250 ft per min and above. The manufacturers of this type of machine claim that it is the outcome of a general tendency toward simplicity in design with efficiency in operation. The machines are generally located over the hatchway. The car is supported by cables which lead from the car directly over the driving-sheave, with overhead installation, then partially around the auxiliary idler or leading-sheave and again over the driving-sheave to the counterweight. With this arrangement a complete turn around the driving-sheave and the idler-sheave is obtained, giving sufficient tractive effort to drive the car. The machine being placed overhead, the cables can lead directly to the car and counterweights; and as this allows the cables bend in the same direction, it is claimed by the manufacturers that s an advantage and that the life of the cables is appreciably lengthened. cial hitches are used for connections to the car and counterweight to nteract the twisting effort due to the reaving of the cables. As soon either the car or counterweight is obstructed, the tension in the cables is reased and consequently the tractive effort reduced. This arrangement, s claimed, brings either the car or counterweight to rest and prevents ning by the limits of travel, and into the overhead beams, should either mber land on the buffers at the bottom of the shaft. Underneath both car l counterweight are placed oil-buffers designed to bring the car or counter-ght to rest at the limits of travel, from full speed. At the top and bottom he hatchway the car is stopped automatically by a series of electric switches. e operation of these switches is so timed that the car is brought to a smooth l gradual stop. The slow-speed shunt-motor, with its control, makes a flex-system. The acceleration and retardation may be arranged to suit the ticular service-requirements. For speeds below 450 ft per min, it is the prac-to obtain the slow speed by passing the cables around sheaves mounted in



. 1. Roping for to 1 Traction-achine

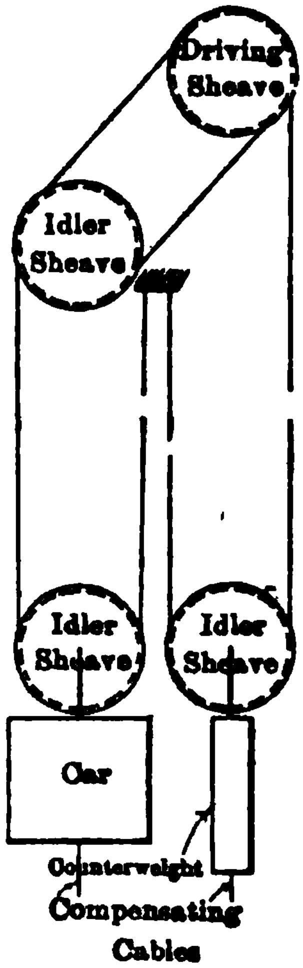


Fig. 2. Roping for 2 to 1 Traction-machine

For full and valuable data relating to the relative advantages of the helical-gear ators as compared with those of the traction-type, see papers published by the H. J. dy Company, Cincinnati, Ohio, and others advocating the geared machines. Editor.

the cross-head of the car and of the counterweight, and anchoring the cab the cables at the top of the hoistway. These sheaves, with their ball bearing are specially made to withstand the heavy service to which they are subject In addition to the above details, elevators of this type should be provided wi all of the regular safety-devices used with passenger-elevators.

Electric Elevators with Push-Button Control. One of the most ingeni and serviceable developments in the elevator-industry is that of the automa electric elevator with push-button control. In New York City this type elevator is permitted only in residences, but in other cities it is used in apart ments, hospitals, and other places where the service is very light and intermitten and it is desired to dispense with an attendant. In the design of these elevat it has been the aim to provide all safety-devices and appliances to make the i stallation absolutely safe, so that the elevators may be operated even by a chi alone, without danger. In each story is located a button, similar in appeara to the ordinary signal-button, and the passenger, by pressing this, may call t car, if it is unoccupied or not in use, to any story. The car comes to the st at which it is required, and stops automatically. When it comes to rest in th story, the entrance-door to the hoistway is automatically unlocked, and it then possible for the passenger to open the door and the car-gate, and enter t

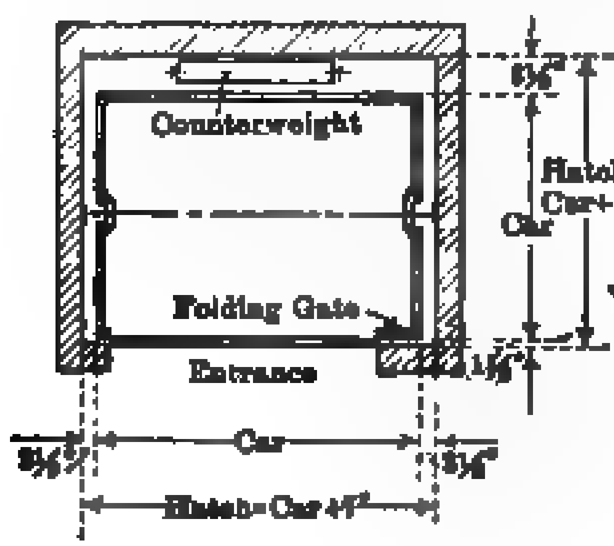


Fig. 3. Standard Hatchway and Car-platform. Side-guides

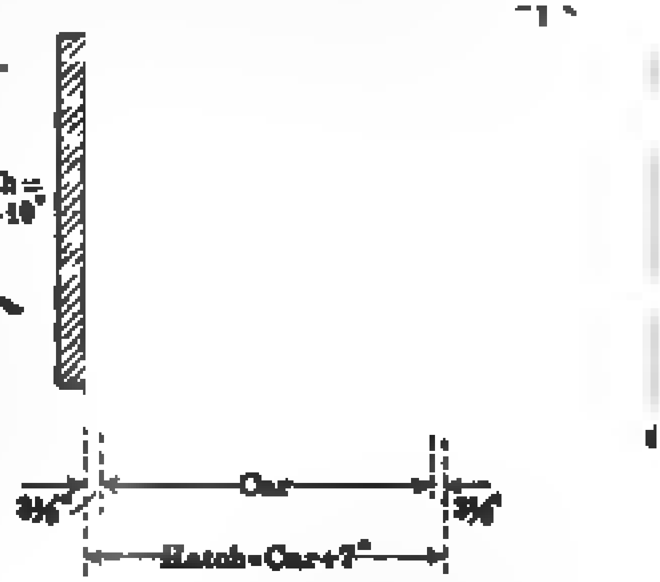


Fig. 4. Standard Hatchway and Car-platform. Corner-guides

car. The hoistway-gates and the car-gate are so arranged that the machine inoperative until both are tightly closed. The hoistway-doors can be open from a hall, only when the car is at the landing of that particular hall. In t car is a bank of buttons corresponding to the various stories served, and al a stop-button or emergency-button. After entering the car, and closing t hatchway-door and the car-gate, the passenger can push the button in th car corresponding to the story to which he desires to go. The car will proce to the designated story and stop automatically. Should the passenger desi for any reason, to stop the car at any point of its travel, he can do so by pushi the stop or emergency-button. The car is in the complete control of the pa senger, as, after the initial operation of calling or sending it to a landing, further operation cannot be interfered with until after both the hatchway-d and the car-gate are opened and closed. This means that no other perso call the car until after the passenger has reached the desired landing, left car, and closed the gate and door. In some equipments for elevators of this ty the device for releasing the door-lock is prevented from operating while the is in motion. This is a very desirable safety-feature, as otherwise each

mporarily released as the car passes up or down the hoistway, and a person i landing can open the door during the momentary period that it is unlocked. ome cases the gate on the car is omitted; but this is a very dangerous prac- and should not be permitted. Elevators of this type are designed for oper- ı with direct current or alternating current, and single or multiphase ıits. Single phase should be avoided, if possible, and before deciding upon type of current, the consent of the electric power company should be ob- ɜd for placing upon their lines a motor with the heavy inrush of current ired at starting.

tandard Relations of Hatchway, Platform and Car-Sizes. (See, also, : 1675.) In Figs. 3 and 4 are shown some typical elevator-layouts for ric installations, with side and corner-posts and steel construction. (See, Fig. 7.) The clearances shown are for elevators traveling at a speed of ft per min or more, and may be reduced about 1½ in for elevators of slower d. Some of the minimum dimensions given with Figs. 3 and 4 vary slightly . those given with Fig. 7 and in Table D, page 1676, but agree in the essen- requirements.

B. Electric, Passenger-Elevator Systems *

levator-Development. The object in view in presenting this material is :o discuss all the details of elevator-construction or the mechanical features, to outline the results of a study in connection with the economic division of enger-elevators and an efficient elevator-service for the traffic of the modern nercial or distinct type of office-building. The requirement of such build- is a very ample and adequate elevator-service, not only because the mon- value of the building may otherwise be affected, but in time of necessity, ıring a fire or other panic, the occupants must be readily brought to safety. ng the early development of the sky-scraper the necessity for a proper ıtor-service was partly overlooked, and perhaps not altogether realized, for : of the older buildings suffer from a lack of traveling-facilities, resulting ı inconvenience to the many occupants. The tenants of the upper stories herefore obliged to wait on the up trip of the elevator, and the people occu- ; the lower portion of the building are left behind on the down trip.

ıe Extensive Use of Elevators. To fully indicate the extensive use to ı the elevator has been adopted for passenger traffic in large cities, the in- e of the Borough of Manhattan of New York City is given. There were 14 about 10 000 machines in service, twice the number that were in oper- in 1904, and these were divided among the different classes of buildings ximately as follows:

5 000 elevators in office-buildings over 10 stories high.
1 500 elevators in office-buildings under 10 stories high.
500 elevators in loft-buildings.
700 elevators in residences.
800 elevators in apartment-houses.
500 elevators in department and other stores.
1 000 elevators in hotels, clubs, institutions, etc.

he matter in Section B of this article on Elevators is, by permission, condensed lapted from data contained in papers by M. W. Ehrlich, consulting engineer. The first appeared in the April, May and June, 1914, issues of Electrical Engineering, fterwards were published in condensed form in Lefax, by the Standard Corpora- f Philadelphia. Section B includes a brief outline of elevator-development, some nic considerations and some installation-data, and the paragraphs of this Section be read in connection with those of Section A, page 1580, and the data compared.

Besides these passenger-cars, the buildings requiring freight-service in an additional 10 000 machines.

Two Common Types of Elevators. In modern elevator-practice are but two common types of successful machines in use, the hydraulic and tric elevators. These may both be subdivided in the classification acc

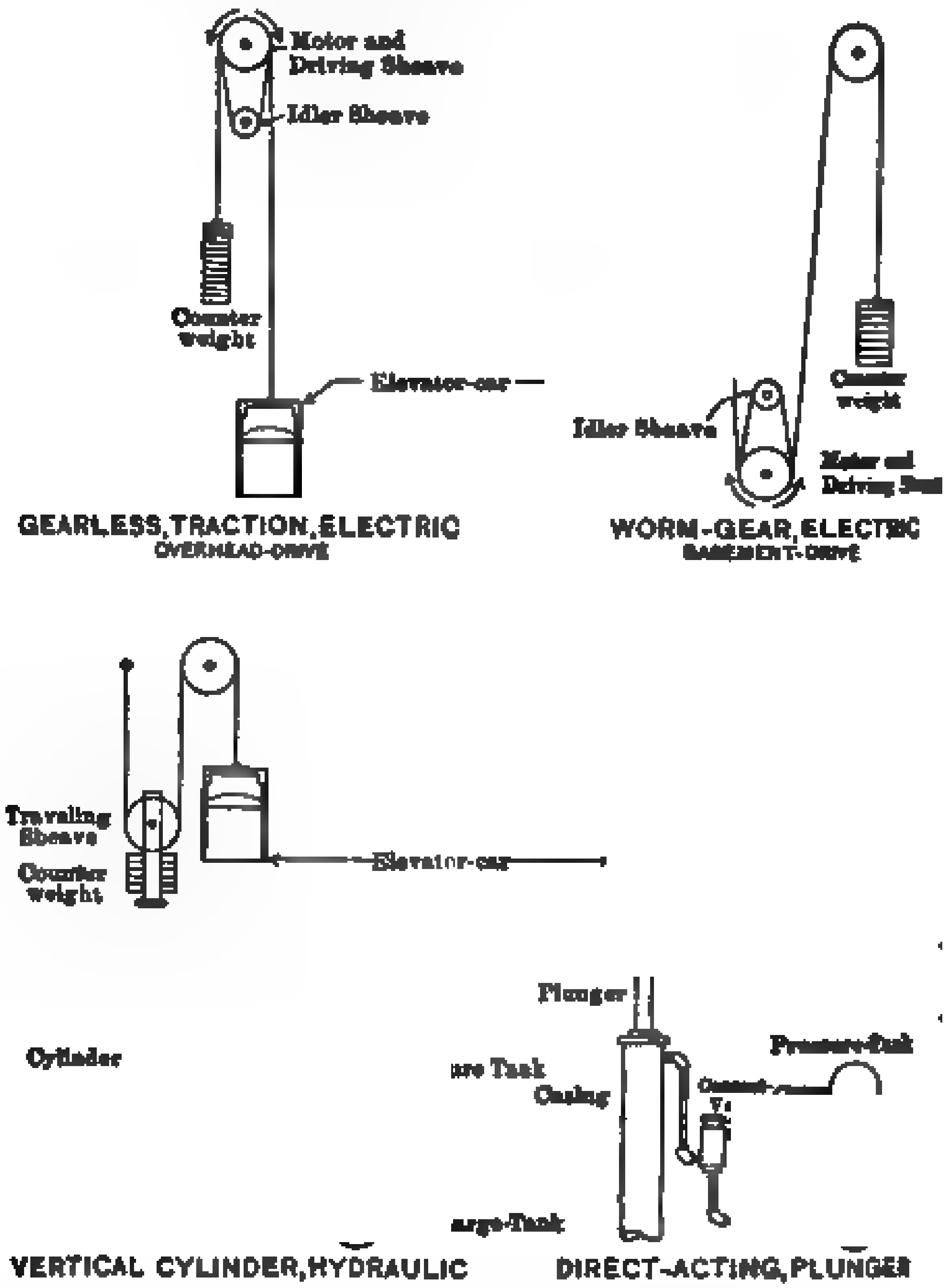


Fig. 5. Some Types and Varieties of Elevators

to the mode of drive or operation and the transmission of power, thereby an apparent variety of elevators. The machine of the hydraulic type of the vertical-cylinder pattern or of the plunger-type, while the electrical ratus may be of the drum, worm-gear or gearless traction-type. Some types and varieties are illustrated in Fig. 5. (See, also, Figs. 1 and 2, and general classification on page 1659.)

A Short Historical Account of the Development of the Com Passenger-Elevator brings one back a little more than half a century

oduction of the first STEAM-ELEVATOR. This form of drive was soon replaced the WATER-BALANCE type of hydraulic elevator, which, even though a faster hine, proved to be, in operation, quite dangerous. For a number of years type enjoyed the distinction of being the only high-speed apparatus until advent of the VERTICAL-CYLINDER HYDRAULIC ELEVATOR, about twenty s later. Running-speeds as high as 400 ft per min were readily attained, on account of the ease in handling and the safety in operation, these ele- rs soon gained favor and were the only types of machines installed in the tall buildings. The electric DRUM-MACHINE made its first appearance in York during the year 1889, and owing to the merits of this new system, electric machine soon established itself as a successful competitor with the raulic type. The only obstacle remaining was to overcome the slower d, and this brought out the Sprague LONG-SCREW ELECTRIC ELEVATOR. ators of this type proved quite costly to maintain and operate, but on unt of their possibilities of speed and high rise, were installed in several structures. These different types of elevators helped considerably in the lopment of the sky-scraper buildings, and as further building projects ght on an extension in height, a hitherto unknown condition of passenger- ator service had to be met. About the year 1900 the DIRECT-ACTING GER HYDRAULIC ELEVATOR was introduced to fulfil this increasing demand ontinued high rise with high speed. The inherent safety in operation and relatively high economy allowed for no doubt as to the possibilities of the GER, but after several years, experience pointed out that the advantages he hydraulic plunger-elevator were somewhat limited in certain directions, only under conditions of a rise not exceeding 150 ft could the character- of the safe and economical plunger-elevator be maintained.

action and Geared Elevators. (See, also, page 1659.) Recent experi- s conducted to perfect an electric elevator that would meet the growing irements of heavy passenger traffic in the newest form of tall office-build- have resulted in the production of what is now commercially known as the TO-ONE, OR GEARLESS TRACTION-ELEVATOR. Among the earliest New York llations of this type of electric elevator may be named those in the Singer ding and Tower, and later those in the Metropolitan Building and Tower, the latest developments include the Woolworth and the Equitable Build- The apparatus used in the Municipal Building is one in which the ines are an adaptation of the usual double-worm-and-gear drive between atively high-speed motor and a cable-drum, a double set of intermeshing gears being employed between the two gear-shafts. In summarizing, it t be well to mention that the commercial or useful life of an elevator and mbined mechanisms seldom exceeds fifteen years, and that where remod- has been resorted to, the ELECTRIC DRUM and WORM-GEAR TRACTION have ly been substituted for the HYDRAULIC TYPE in buildings not exceeding twelve to sixteen stories in height; and that in higher structures the ss traction-elevator or its modification in the form of an electric TWO- E TRACTION-ELEVATOR has been resorted to.

fety of Electric and Hydraulic Elevators. (See page 1664.) It is however, that both the electric and hydraulic types of elevators have perfected to a state of high efficiency, and they may, therefore, be used entire safety. Of the hydraulic types it may be said that the plunger- ors are inherently safer than those which are suspended, or than even the modern electric traction-elevators; but it cannot be denied that the many ments and improved appliances attached to elevators of the various electric have made the latter as reliable as hydraulic machines designed according

to best practice. It is claimed that the electric traction-elevator is [illegible] free from the element of danger because of the improved methods of [illegible] transmission and the peculiar form of windings used for the drive.

Comparison of Merits of Electric, Traction, and Hydraulic Pl[illegible] Elevators. In narrowing down the question as to the merits of the ELE[illegible] TRACTION-ELEVATOR and of the HYDRAULIC PLUNGER-ELEVATOR for pass[illegible] service in tall office-buildings of to-day, it might be well to note that the elevator-installations, almost without exception, have favored the el[illegible] Not only is the cost of installing the traction-machine from 25 to 35% les[illegible] that of the plunger-type, but the room occupied by the driving-machine [illegible] reduced to a minimum, and, as a matter of fact, may be placed at the head [illegible] directly over the elevator-shaft. If no local supply of electricity is availab[illegible] the premises, the public source may be resorted to. The difficulty with [illegible] plunger-elevator for high-rise, high-speed work lies in the requirement for [illegible]ing the mass of water and the massive plunger proper, and as this [illegible] weight cannot be readily and smoothly stopped, the result is a sluggish [illegible] starting and stopping. At any rate, it remains an open question as to wh[illegible] the economic values attached to modern buildings would favor the install[illegible] of the plunger-elevator, with its accompanying pumping-plant, which n[illegible]sarily occupies considerable floor-space. The choice, therefore, would [illegible] favor the HIGH-RISE, HIGH-SPEED ELECTRIC TRACTION-ELEVATOR for pass[illegible] service. (See, also, paragraph on Electric Versus Hydraulic Elevators, [illegible] 1660.)

Table A. Relative Operating-Costs of Elevators

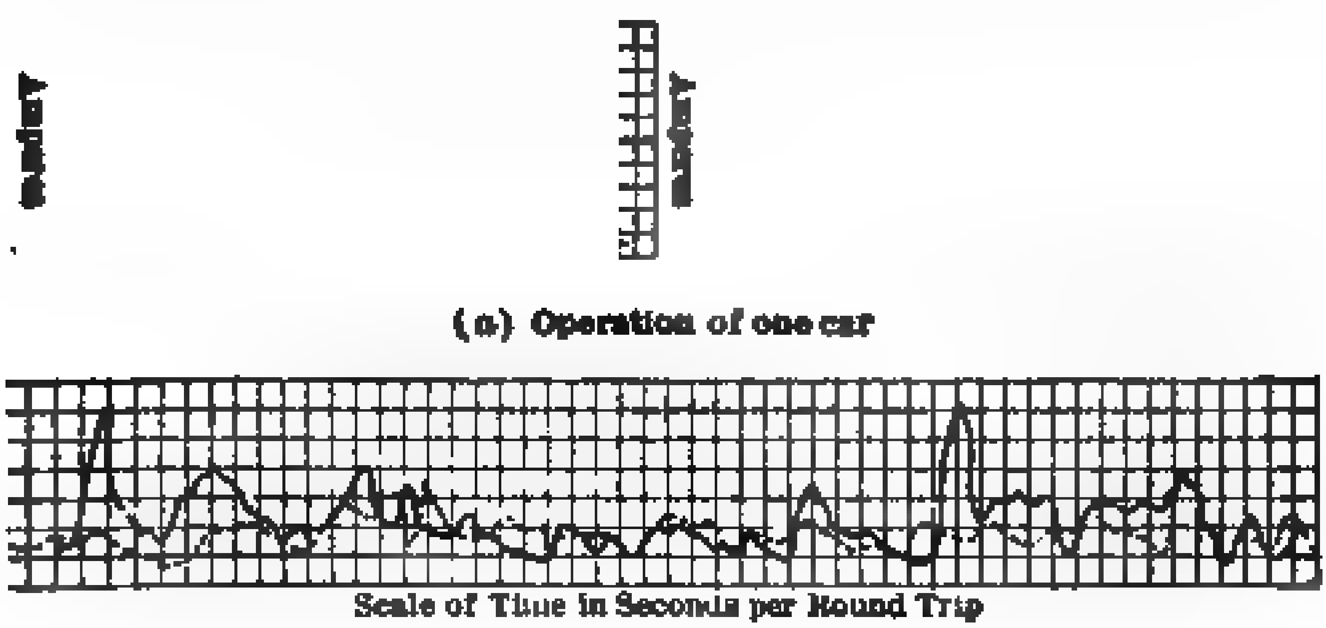
	Office-building				Loft-building				Apartment-[illegible]		
	Traction electric	Worm-gear electric	Vertical-cylinder hydraulic	Direct plunger	Traction	Worm-gear	Hydraulic	Plunger	Traction	Worm-gear	Hydraulic
[illegible]ls	85	72	68	65	80	68	[illegible]		8[illegible]	60	55
Cents per car-mile	25	22	20	19	23	20	1[illegible]			18	17
Dollars per car per annum	2 100	1 890	1 680	1 600	2075	900	[illegible]		60[illegible]	510	[illegible]
Per cent of all operating-costs . ..	14.1	12.0	11.3	11.0	18.0	15.4	1[illegible]		6[illegible]	12.0	11.[illegible]

Relative Operating-Costs of Elevators. The figures given in Tab[illegible] may prove of interest in pointing out the relatively higher operating-costs [illegible] different ELECTRIC types over the VERTICAL-CYLINDER HYDRAULIC and PLU[illegible] ELEVATORS. The values given represent only the cost of labor, power, re[illegible] and supplies. By a close perusal of the amounts listed, it will be con[illegible] that the economies of the plunger cannot be utilized beneficially in tall [illegible] buildings, on account of the mechanical difficulties, and in other types of [illegible] buildings, allowing for a low rise, the installation cost becomes exorbitant[illegible] the relatively high first cost of this type of machine were taken into consid[illegible]tion, with an addition for the extra cost in building-construction necessar[illegible] the space occupied by the pump and tank-equipment, the total expendit[illegible] the whole would show no great favor either way. In explaining the v[illegible] given in Table A, it should be understood that the figures are computed [illegible] basis of actual records of several buildings that have been brought to the [illegible]

e. The general method of comparing records in business buildings is to are the costs with the total annual income or rental. The total oper- -costs include the expense in the mechanical, electrical and building rtments, covering all costs of labor and material for the maintenance of the ent divisions of service. Therefore the annual cost of operating an ele- :-system is given as a percentage of the gross rentals received, and is er stated as a percentage of the total operating-expenditure of the build- under consideration. The average cost in cents per car-mile traversed is given, together with the average annual cost in dollars to pay for the ' of operating and repairing, the necessary power, and the material and lies required per single elevator.

onomic Considerations. The efficient operation of an elevator-system not rest altogether on the economic division and disposition of the cars, as uman element becomes one of the main factors. It is self-evident, there- that the service of an elevator is limited not only by the different classes of ngers entering, riding and leaving the conveyance, but by the experience e hallman or STARTER and his ability to understand the demands of the c and the personal peculiarities of the elevator-operators.

me-Schedules. It is now common practice to dispatch the various ma- s of an elevator-system on a predetermined time-schedule, thus avoiding great extent any confusion or overcrowding that would otherwise arise. s been well established that the travel of elevators under consecutive-trip lule-operation allows for a highly efficient service, not only in the handling e traffic, but in the demand for power, which is thereby reduced to a min- a.



(a) Operation of one car

(b) Operation of a bank of elevators

Fig. 6. Recorded Current-consumption of Gearless Traction-elevators

wer-Diagrams. The POWER-DIAGRAMS (Fig. 6) point to the effect of a and a proper service under different conditions. The upper curve (a) was under test-conditions and represents the operation of one elevator. The in the single car is approximately equal to the designed machine-balance, on the up and down trips, and the number of stops corresponds to the ge per car per mile under actual service in office-buildings. This diagram en mainly to allow for a proper understanding of the combined curve (b), ing the actual round-trip operation of a bank of elevators in one of the New sky-scraper buildings at an early-morning hour. The full or solid-line shows an excessive power-demand due to an inconsistent SCHEDULE, the

cars having been dispatched by a starter who may be identified as X, whi dotted or broken-line curve shows the more expert handling under the conse trips by starter Y, the same operators running the cars in each case.

Safety-Appliances. (See, also, page 1664.) To minimize the many dents in elevator-practice, a SAFETY-LOCK is recommended, so attached will not permit the elevator to leave a landing until the gate has been Accidents are seldom, if ever, due to the faulty behavior of the elevator but sometimes the breaking of suspension-ropes, as recorded by a relatively cases, will result in a serious accident. The most frequent cause of acci connected with the operation of elevators, is that due directly to the gence of the operator in handling the doors or elevator-gates, and this avoided by the installation of the safety-locks above recommended. as has been practically demonstrated, many of the safety-appliances older installations designed to stop a falling elevator have usually failed but the improved wedge-type of JAW-SAFETY, actuated by a SPEED-GOV and attached to the more recent installations, usually acts when the exceeds its normal running-speed. This generally occurs when the design safe-distance limit has been passed, and the jar occasioned by the final of the car is not altogether a pleasant experience. The serious injuries fatalities due to the falling of an elevator are proportionately small when pared with the entire list, and amount to about 20% of the total, whereas loss of life caused by open and unlocked gates in elevator-practice accounts for the remaining 80%. The only safety-device, therefore, that be called useful, as it eliminates the personal element, is a SAFETY-LOCK the several automatic devices now available for this provision of safety, serve merit; and while some are purely mechanical, others are actuated trically, and only by the installation of such automatic locks will unnece elevator-accidents be avoided.

Signal-Systems. A SIGNAL-SYSTEM is essential to an efficient service. matic electric-light indicators at the different landings, with a mecha indicator on the ground-floor or street-landing, will be found highly eff even though not the simplest. Briefly described, the system is composed dynamotor supplying current for the magnets, push-buttons and lamps. each landing one or more sets of push-buttons are arranged for both the UP DOWN signal, and over each elevator-gate two lamps of different color, one another, to indicate the direction of car-travel; and each elevator-car is provided with a signal-lamp and a transfer-switch or push-button. A mech ical indicator on the main landing informs the starter as to the location of different elevators, and thereby aids him in exercising full discretion as to to dispatch the next car. The general system operates in a manner appr mately as follows: When a push-button is pressed for either direction story, it actuates a magnet corresponding to that story, which in turn sig to the operator in any approaching car, thereby indicating a waiting passe and, according to the movement of the elevator, further contact is made the outside signal-lamps at that story showing to the waiting person the approaching that floor. In a properly proportioned elevator-system the tr fer-switch is seldom used, but in buildings in which the travel becomes over during the rush-hours, and when an approaching car is filled to its cap the operator may press the transfer-button and thereby signal the car follo

Traffic-Capacity of Elevators. The TRAFFIC-CAPACITY of an elevat its passenger accommodations must necessarily be of such proportions handle the travel of the tenants of the building and also of their visitors sure a proper working schedule. From a study of existing systems in which

ator-service is considered adequate, it is found that the questions of BUILD--OCCUPANCY as related to BUILDING-AREA and elevator TRAFFIC-CAPACITY be combined into a consideration of a proper UNIT AREA for the elevator. egard to the determination of the MAXIMUM TRAFFIC-CAPACITY of a passenger-ator, experience shows that an average weight of 140 lb may be allowed for passenger, and as each size of car has its corresponding load at the rated d, the total load divided by 140 gives the maximum number of passengers an ator can accommodate at its designed speed. In another simple computa- for this result, an allowance of 2 sq ft of car is made for each passenger. The imum capacity of an elevator may be of interest in computing the time re-ed to empty a building in case of emergency; but when a car of proper unit is installed, this condition is taken care of. Tests have shown that the RAGE PASSENGER TRAFFIC of an elevator-system bears a definite relation to TENANCY of the building, and to the MAXIMUM TRAVEL, the result being that essed in Formula (6).

umber of Elevators. (See, also, page 1661.) Modern practice tends to that the NUMBER OF ELEVATORS required for any office-building is really rned by the physical aspects and conditions of that building. Whereyer it t practicable to use a car of large area, the number required will certainly n excess of that necessary when large cars are used. It is not advisable, efore, to base any conclusions on the number of cars to adequately satisfy rtain condition, unless the UNIT AREA OF THE CAR is considered.

ocal and Express-Elevators. Another important consideration is the ion so common in high-class office-buildings, namely, the proper service of L and EXPRESS-elevators.

ormulas for Elevator-Service. The formulas given below are well sub-tiated, and give economical service-conditions based on existing systems in larger cities of the United States. By these formulas the number of eleva- required, the division of service, and their operation may be determined.

$$E = A/24\,000 \qquad (1)$$

$$f = n/2 + 2 \qquad (2)$$

$$Te = (25/s + 5/100)\,n \text{ and } Tl = (25/s + l/10)\,n \qquad (3)$$

$$Me = 2\,n/7\,Te \text{ and } Ml = 2\,n/7\,Tl \qquad (4)$$

$$Ce = 115\,n/100\,Te \text{ and } Cl = 115\,n/100\,Tl \qquad (5)$$

$$pe = 300/Te \text{ and } pl = 300/Tl \qquad (6)$$

notations in the formulas are:

E = number of elevators required
A = square feet of gross building-area served
f = story at which express-run terminates
n = total number of stories served
s = speed of elevator, in feet per minute
Tl = local round-trip time, in minutes
Te = express round-trip time, in minutes
Ml = miles traveled per hour by local
Me = miles traveled per hour by express
Cl = current consumed per hour by local, in kilowatt-hours
Ce = current consumed per hour by express, in kilowatt-hours
pl = passengers carried per hour by local, one way, up or down
pe = passengers carried per hour by express, one way, up or down

e figures in Table B represent the AVERAGE LOAD AND SPEED-COMBINATIONS rious heights of buildings, together with the usual AREA OF THE ELEVATOR-

CAR consistent with the standard sizes manufactured, and should be [illegible] basis for selecting the proper unit areas in connection with Formula [illegible] many factors entering into the operation of an elevator would affect the c[illegible] consumption to a considerable extent, as may be seen in Fig. 6, previ[illegible] plained. But Formula (5) agrees with modern service under average ope[illegible] conditions.

Table B. Unit Area, Load and Speed-Combinations

Number of stories	Car-area, sq ft	Load, lb	Speed, ft per min
8 to 13	25	1 700	250 to 350
14 to 22	30	2 000	350 to 600
23 to 30	40	3 000	400 to 600

Table C. Elevator-Installation Data

1	2	3	4	5	6	7
Building		Number of elevators required				
Number of stories	Gross area, sq ft	Total car-area, sq ft	Cars at 25 sq ft	Cars at 30 sq ft	Cars at 40 sq ft	By Formula
8	80 000	89	4	....	....	4
10	100 000	111	4	....	....	4
12	120 000	133	5	....	....	5
14	210 000	262	11	9	....	9
16	240 000	300	12	10	....	12
18	270 000	337	14	11	....	11
20	300 000	375	15	13	10	13
25	375 000	577	....	19	15	16
30	800 000	1 221	....	40	30	35

	8	9	10	11	12
Number of stories	Round trip time in minutes				*f*, or expressed in stories
	Tl at 350 ft per min	*Tl* at 500 ft per min	*Te* at 500 ft per min	*Te* at 600 ft per min	
8	1.3	....	....	....	....
10	1.7	....	....	....	....
12	2.0	....	....	....	....
14	2.4	2.1	....	....	....
16	2.7	2.4	1.6	....	13
18	....	2.7	1.8	....	11
20	....	3.0	2.0	1.8	12
25	....	....	2.5	2.3	15
30	....	....	3.0	2.7	17

Installation-Data. In order to facilitate the ready understanding of the rious formulas given, Table C, embodying the computations, is presented. ne various headings included are numbered in respective order from 1 to 12, that an explanation of the items considered will not be confusing. Under lumn 1 is listed the heights of buildings, with the assumed floor-areas, extend- g the full height of the structure, given in column 2. In column 3 are listed e actual square feet of car-area now provided in many buildings of similar or-space and with an adequate service. This is intended as a guide where e considerations in planning the building have included a means of accom- odating the standard-sized elevators most suitable for that building and where rious attention has been given to the disposition of the cars. But, on the ner hand, the values listed may also be used to advantage in proportioning e number of elevators required under any conditions, and where the physi- aspect of the building does not allow for an economic disposition of the vators. Any conservative unit area best suited to the conditions may then allotted for each car, and the number of elevators then determined. Col- ns 4, 5 and 6 give the numbers of cars for various standard unit areas, while e values in column 7 are computed by Formula (1).

The Local and Express Round-Trip Time for different running-speeds given in columns 8, 9, 10 and 11 of Table C, and the value for f as given Formula (2) is given in column 12. It will be noticed that in columns 8 d 9 the time occupied in traversing the heights of buildings exceeding eighteen ries is slightly more than would actually prove economical. It might be well, refore, to point out that the speeds of local elevators for high buildings might increased to advantage; but whether the service is local or express, it is not visable to exceed a speed-rate of 600 ft per min. In order to rectify this con- ion, under the speeds considered, the number of express-elevators must then more than half the total number in the system, and a subdivision of express- vice proper is also necessary. (See, also, Table Showing Number of Eleva- s Required and notes following, page 1661.)

Sizes of Hatchways and Car-Platforms. (See, also, page 1661.) The es of elevator-car platforms and hatchways of unit areas heretofore con-

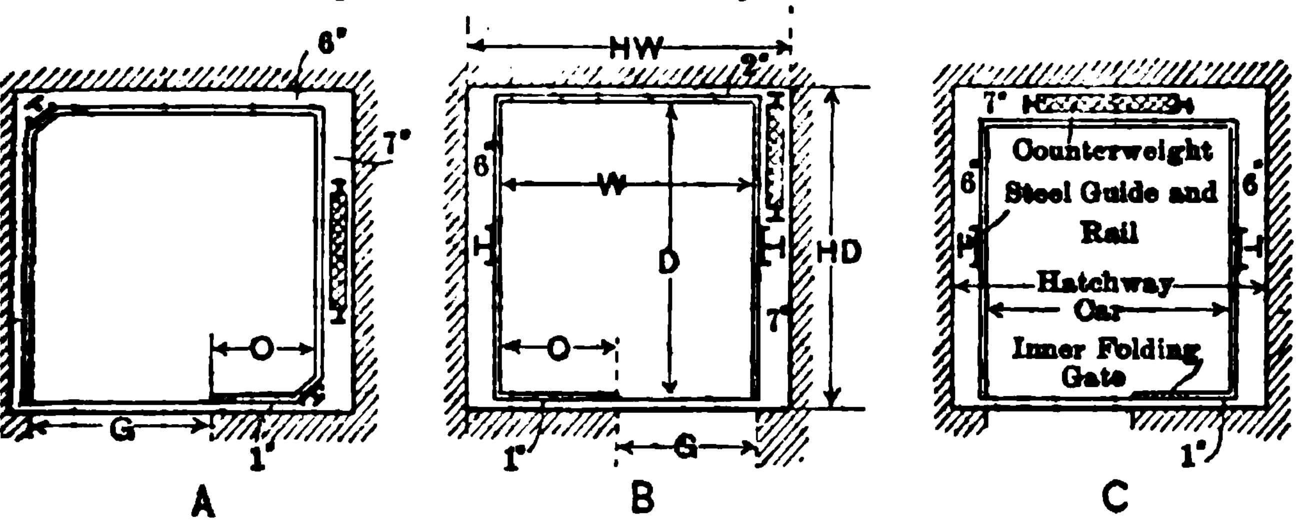


Fig. 7. Typical Layouts for Elevator-hatchways and Car-platforms

red are shown in the following diagrams (Fig. 7) illustrating three typical ns of modern installations with steel guide-rails. (See, also, Figs. 3 and 4.) gate or door-opening may be either right-hand or left-hand, as best suited lanning, structural, or other conditions. The clear inside dimensions of the ssary hatchway are given, and also the clearances required between this and car. Some of the minimum dimensions given with Fig. 7 and in Table D r slightly from those given with Figs. 3 and 4, page 1666, but agree in the ntial requirements.

Table D. Sizes of Elevator-Car Platforms and Hatchways

Dimensions	Area of car-platform					
	25 sq ft		30 sq ft		[illegible] sq ft	
	ft	in	ft	in	ft	in
W = inside width of car	6	0	6	3	7	[illegible]
D = inside depth of car	4	3	4	9	5	[illegible]
O = space for operator	2	3	2	3	2	[illegible]
G = gate-opening	3	9	4	0	4	[illegible]
HW = hatch-width, car A	7	0	7	3	[illegible]	[illegible]
car B	7	4	7	7	[illegible]	[illegible]
car C	7	3	7	6	[illegible]	[illegible]
HD = hatch-depth, car A	5	1	5	7	6	[illegible]
car B	4	9	5	3	6	[illegible]
car C	5	2	5	8	6	[illegible]

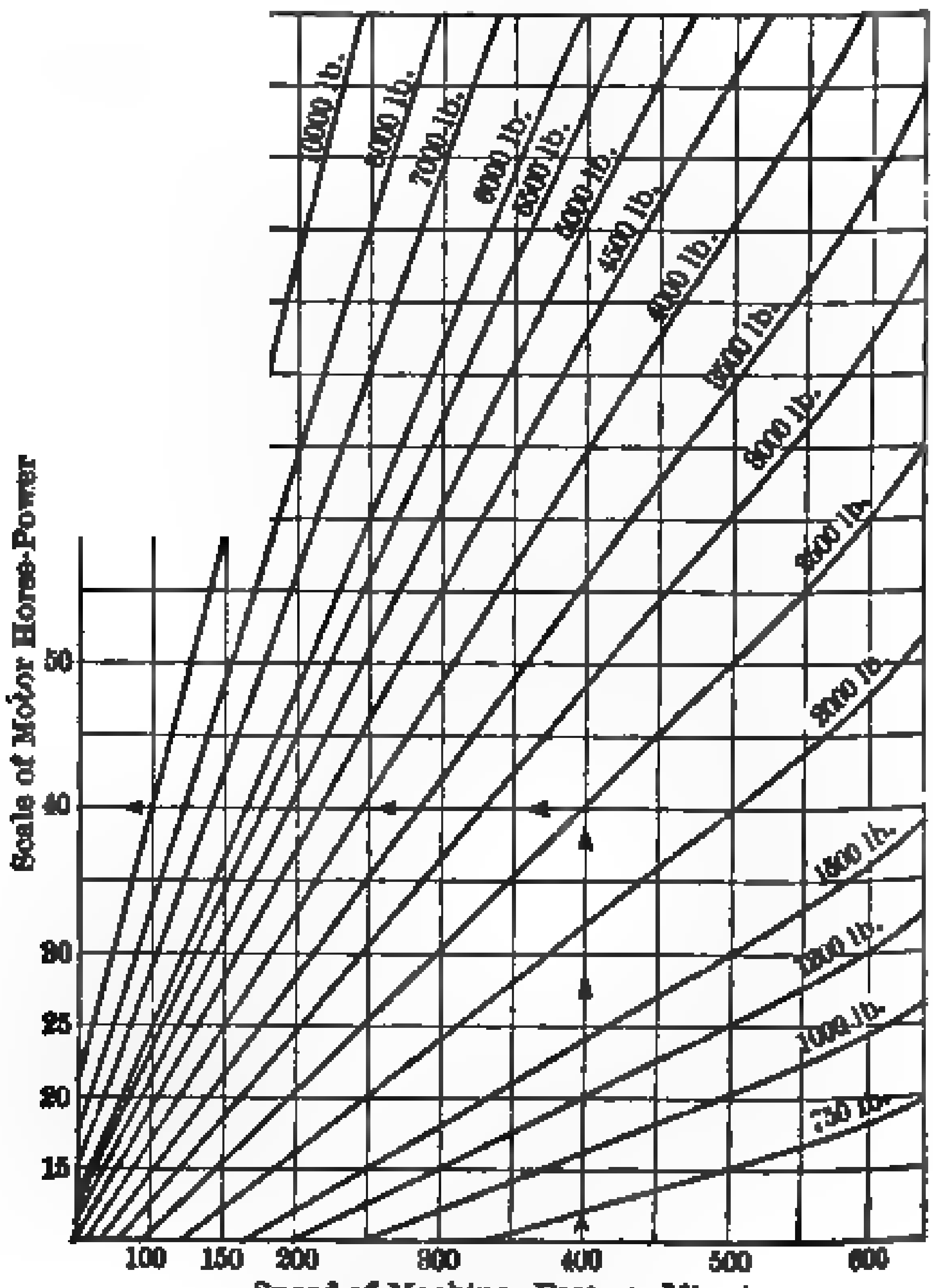


Fig. 8. Motor-sizes for Electric Elevators

Size of Motor. It is often helpful to be informed as to the SIZE OF MOTOR re-
uired for an installation, and the diagram (Fig. 8) may be used for this purpose.
or sake of illustration in the use of the diagram, a speed of 400 ft per min is
sumed, with a combined load of 2 500 lb. Following the line marked with an
row from the speed of 400 ft, the point of intersection is then at 2 500 lb. From
is point follow the line as indicated to the scale of motor-sizes, and the result
about 40 horse-power.

Table E. Current-Consumption

Motor-size	Starting-current	Running-current
20 horse-power	102 amperes	74 amperes
40 horse-power	202 amperes	147 amperes
60 horse-power	292 amperes	213 amperes

Current-Consumption. Table E gives the CURRENT-CONSUMPTION of motor-
es common in elevator-practice. The figures are for direct-current motors
erating at 230 volts and are based on the results of tests.

Electric Feeders. To aid in the selection of well-proportioned ELECTRIC
EDERS for elevator-motors, Table F is given. The figures are for 230-volt,
ect-current machines.

Table F. Wire and Conduit-Sizes for Electric Elevators, 2-Wire, 230-Volt, Direct-Current Systems

Motor-h.p.	Wire			Maximum run or distance for 2% drop, ft	Conduit		
	Size of each wire		Underwriters carrying capacity, amperes		Trade size for 2 wires	Inside diameter, in	Outside diameter, in
15	No.	3	80	154	1¼	1.38	1.66
20	No.	1	100	174	1½	1.61	1.90
25	No.	0	125	186	1½	1.61	1.90
30	No.	00	150	198	2	2.06	2.37
35	No.	000	175	212	2	2.06	2.37
40	No.	0000	225	226	2	2.06	2.37
45	No.	0000	225	226	2	2.06	2.37
50	300 000	c.m.*	275	248	2½	2.46	2.87
55	300 000	c.m.*	275	248	2½	2.46	2.87
60	400 000	c.m.*	325	272	3	3.06	3.50

* Circular mils.

MAIL-CHUTES

General Description. This system of mailing letters by means of a specially
structed chute connected with the receiving-box at the bottom, has come
such general use in public buildings, office-buildings, apartment-houses
hotels, that the restrictions affecting the same and what is required in the
of preparation should be known to architects. The system is installed by
patentees, under regulations of the Post-Office Department governing its

construction and location, and for this reason it is well to consult the make before permanently locating the apparatus on the plans. It may be plac any building of more than one story, used by the public, where there is a delivery and collection-service, in the discretion of the local postmaster, sub to whose approval the contracts are made.

The Chute and Receiving-Box. The chute is required to be made with removable front and a continuous, rigid, vertical support is absolutely necess It must be of metal, its front must be of plate glass, and it must bear the insc prescribed by the department; and the whole apparatus, when erected and Government lock put on the box, passes under the exclusive care and contr of the Post-Office Department, and the chutes become a part of the receivi boxes. These boxes may be of various patterns and highly ornamented and a furnished by the makers in connection with the chutes. The work of prepari a rigid support for the chute and cutting and finishing the openings in the floo is of the utmost importance, and details showing the usual arrangements a always given.

Preparatory Work. The requirements for what the manufacturers ca PREPARATORY WORK include a flat, vertical, continuous surface not less tha

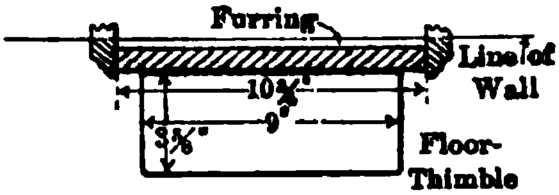


Fig. 1. Wooden Support for Mail-chute

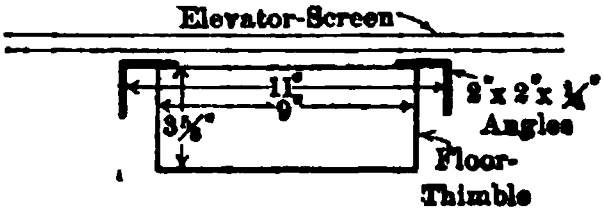



Fig. 2. Steel Support for Mail-chute

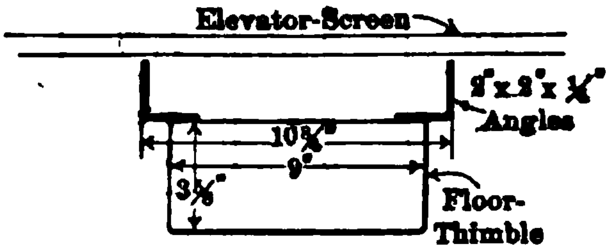


Fig. 3. Alternate Steel Support for Mail-chute

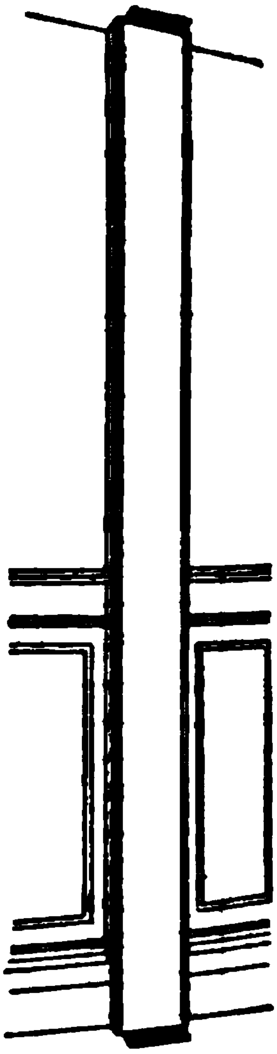


Fig. 4. Preparatory Work Complete for Mail-chute

10½ in wide, extending from the floor of the ground-story to a point 4 ft 6 i above the finished floor in the top story, and an opening in each floor direct in front of and centered upon this surface. These openings are neatly finishe and their size and shape determined by setting in them thimbles of iron whi

* The Cutler Mail Chute Company, Rochester, N. Y.

re furnished and delivered by the patentees, as part of their contract. In ordinary installations a casing of wood, suitably molded and finished to match the trim of the building, answers every purpose. Such a casing is shown in plan, Fig. 1, with the opening finished by the iron thimble. In buildings, or sometimes in a few stories, where a more elaborate finish is desired, marble is substituted for wood, the form and construction of the casing being adapted to the material, but of course without disturbing the size and form of the front surface. Steel angles are used where the use of wood is objected to, or where it is necessary to run the chute in front of an elevator-screen, or in other locations where a solid wall is not available to support the casing. Steel square-root angles, 2 by 2 by ¼ in in section, are generally used, and set as in Fig. 2, but sometimes, where it is desirable to fill up the space between them and the elevator-screen, they are reversed, as in Fig. 3. The angles are usually bolted to the beams, and in any case must be straightened so that they are without twists or kinks, and the surface which receives the mail-chute plumb and flush in all stories. Fig. 4 gives a general view of the mail-chute casings and floor-openings ready to receive the chutes themselves. This work of preparing the building, except the cutting or leaving ready the necessary openings in the floors, is now usually included in the mail-chute contract, as it has been found for many reasons undesirable to separate it. The necessary openings in floors, and all patching around such openings, should be included in the mason's or other proper specifications.

Essential Points to be remembered are (1) that no bends or offsets can be made, a vertical fall being absolutely essential, and (2) that the entire apparatus must be exposed to view and must be accessible, that is, it is not permitted to extend the work behind an elevator-screen or partition or through any part of the building except a public corridor.

REFRIGERATORS *

General Requirements. The following information is given as a guide to architects in providing for refrigerators in large residences, hotels, clubs, hospitals and other institutions. Consultation with a reliable refrigerator-builder, however, is always desirable before deciding upon spaces to be occupied by refrigerators, refrigerating-rooms, etc., as a satisfactory refrigerator cannot be adapted to a badly proportioned space. (See, also, Design of Refrigerators, under Mechanical Refrigeration, page 1691.)

Residence-Refrigerators. Care should be taken to select a refrigerator which is simple in operation and easily cleansed, as modern sanitary science has traced much illness to faulty refrigeration. Thorough insulation is an important feature in a refrigerator, as upon this depends economy in the use of ice and the securing and maintaining of the low temperature necessary to the proper preservation of food. Fig. 1 shows a kitchen-refrigerator for use of families of ordinary size. The ice-compartment is located in the middle division. The depth should not be more than 3 ft nor less than 2 ft, and the height may vary from 6 ft 6 in to 7 ft. The length of the front largely determines the capacity and should range from about 4 to 7 ft. Fig. 1 shows, also, a most satisfactory method of accomplishing the outside-icing feature which consists of a double outside icing-door complete, with frame and jamb. This is provided by the refrigerator-builder to fit the rough opening furnished by the owner in the outside wall of

* Valuable data and the drawings relating to this subject were furnished the author and editor by The Jewett Refrigerator Company, Buffalo, N.Y. Practical data were furnished, also, by The Brunswick-Balke-Collender Company, New York City. There are numerous other reliable firms whose refrigerator-work has the highest reputation.

the building. With this method a minimum outside opening is required to furnish a maximum inside opening for ice. The DRAIN-PIPES should be as short and straight as possible and should be readily detachable for cleansing purposes. The drain should be properly trapped in the floor of the refrigerator and carried through the floor of the building, discharging over the plumber's open connection as shown in the elevation of Fig 1.

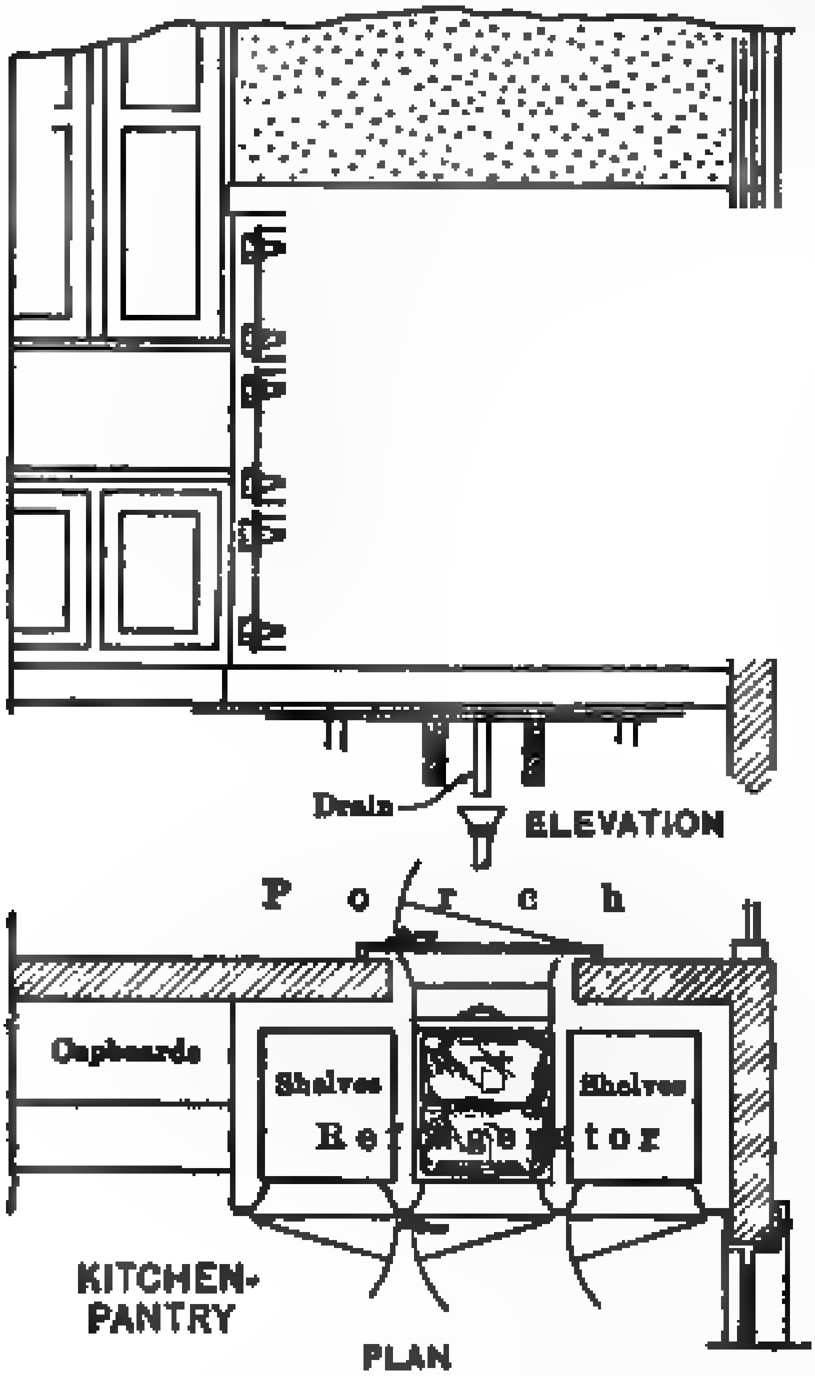


Fig. 1.* Kitchen-refrigerator for Small Family

Fig. 2 shows a refrigerator for use in a butler's pantry where economy of space is important. The ice-compartment is of galvanized steel throughout and is removable for convenience in filling as it slides on roller-bearing runways. When the ice-compartment is replaced in position the outside door closes over it. The adjoining storage-compartment is generally fitted with one removable shelf, below which is a bottle-rack for horizontally placed bottles and a space for standing bottles. The depth should be about 2 ft and the height 2 ft 4 in, under counter-top. The length of the front determines the capacity, but it should never be less than 3 ft. For a double refrigerator with a central ice-compartment and storage-compartments at either side, 5 ft is a convenient length. The exterior finish and hardware should correspond with the adjacent trim. The most sanitary and attractive interior finish for storage-compartments consists of white plate glass for the walls and ceilings and tile for the flooring. The usual complement of refrigerators for use in ordinary families consists of one adjacent to the kitchen and one in the butler's pantry. For large families the number could be the same with the capacity greater.

Refrigerators for Hotels, Clubs, Etc. MECHANICAL REFRIGERATION has largely superseded ICE as a cooling-agent where the refrigerator-equipment consists of several units, as in hotels, clubs and institutions. (See, also, Mechanical Refrigeration, page 1691.) The arrangement of refrigerators is similar to that employed where ice is used, as the refrigerating-coils are often contained in compartments corresponding to ice-compartments; the alternative method is to place the coils against walls of storage-compartments. Refrigerating-coils are generally of 1¼-in pipe, the length of coil depending upon the temperature required. Fig. 3 shows a practical layout for the working-department of

* The Jewett Refrigerator Company.

ɔd-sized club, and illustrates the proper complement of mechanically cooled rigerators, together with adjacent operating-equipment. No. 1, a store-room rigerator, has the front arranged in one full-height door and is fitted with three rs of shelves throughout. No. 2, a meat-refrigerator, is also accessible through ull-height door and is fitted with shelves and meat-racks. No. 3, a broiler d fish-refrigerator, has the front anged in two doors, each door ening onto a series of six galvanized eet-steel pans sliding on self-sustaining roller-bearing runways. No. a serving-pantry refrigerator, is ɔdivided by an insulated partition o three separate and distinct comrtments, those at the left and right ing each accessible through two ors, while the middle compartment accessible through one door, below ich is a series of four drawers ding on self-sustaining roller-bearing nways. The doors open onto reɔvable shelves throughout. No. 5, ice-cream refrigerator, occupies a sition in the serving-pantry counter d has the top arranged in one liftcover. Its interior fittings consist three 20-quart porcelain-lined iceeam jars and one glacé-frame for ncy forms of ice-cream. No. 6, a stry-refrigerator, has the front arnged in four doors, two upper doors ening onto removable shelving, and o lower doors onto pastry-pans ding on angle-iron runways. No. a bar-refrigerator, is subdivided by insulated partition into two separate and distinct compartments, each cessible through four doors. The upper doors open onto three tiers of movable shelves for standing bottles, while the lower doors open onto five rs of racks arranged specially for horizontal bottles. The equipment scribed above will also satisfactorily cover the requirements of a moderate-sized tel.

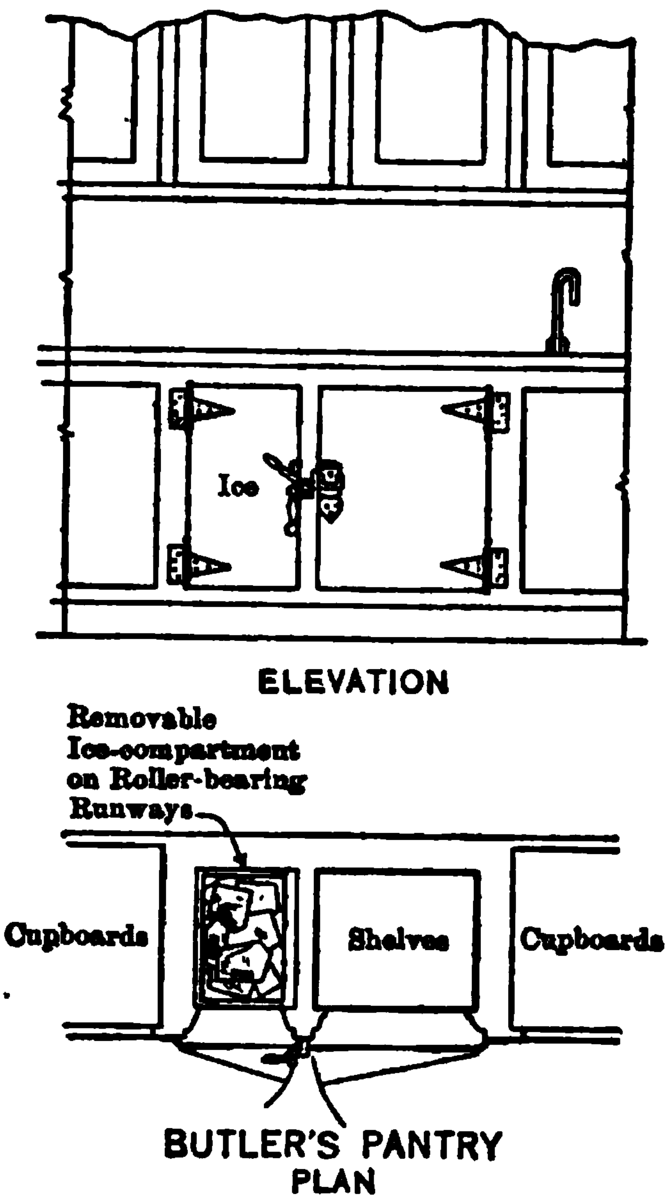
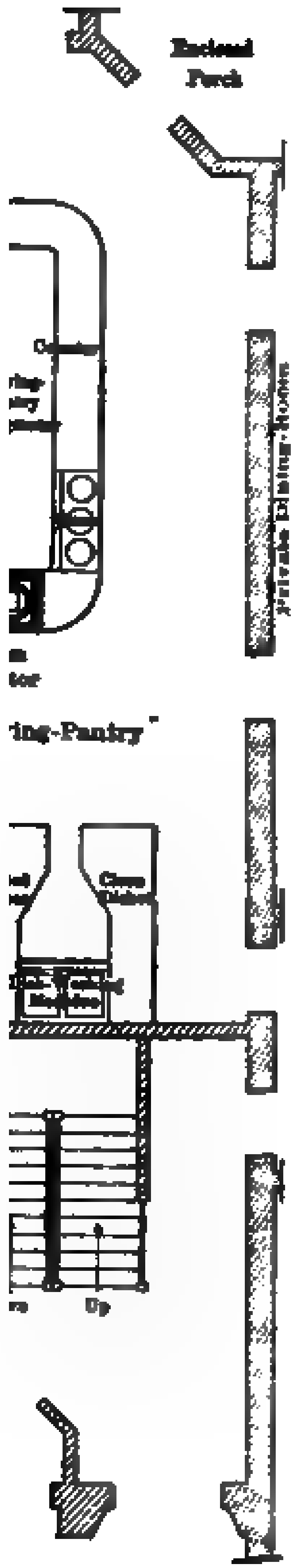


Fig. 2.* Refrigerator for Butler's Pantry

Refrigerators for Hospitals. The usual complement of refrigerators for nall hospitals consists of one large storage-refrigerator, one refrigerator for e chef's use in or near the kitchen, one for milk and butter and one iron-lined est for broken ice. For large hospitals the same number with increased capacity and with the addition of small diet-kitchen refrigerators, and possibly a ortuary-refrigerator for two or three bodies, will meet the requirements.

The Height of Large Refrigerators for hotels, clubs and institutions, to be tered through full-height doors, should be from 10 to 12 ft, if equipped with erhead ice or coil-compartments; with side ice-compartments or coils placed ainst walls, the height should be 7 ft 6 in or 8 ft. The smaller refrigerators,

* The Jewett Refrigerator Company



Main Dining-Room

Fig. 3.* Plan of Refrigerators for Large Club-house

* The Jewett Refrigerator Company.

cessible through half-height doors, hinged covers, drawers, etc., should be aced on a 3-in sanitary cement platform finished with cove to floor of building. nese refrigerators should not be higher than 6 ft 6 in unless provided with overad ice or coil-compartments, in nich case the height should be from to 9 ft.

Insulation. (See, also, The Value Good Insulation, page 1690.) efrigerators in modern hotels, clubs, stitutions, etc., are insulated with overnment-standard corkboard, the rge refrigerators being constructed 4-in cork throughout, in two courses 2 in thickness, and with all joints oken. Cork is applied to adjacent alls of a building with Portland ment, ½ in thick, and this cement used, also, in applying the inner ourse of cork to the outer course in alls, partitions and ceilings. All ork in the flooring is asphalted ater-tight. Interior finish may be Portland cement throughout or of lvanized sheets on walls and ceilgs and of Portland cement on floors. r the walls and ceilings may be of sed-on porcelain or white plate glass, nd the floors of tile, all depending pon the grade and character of the uilding to be equipped. The inlation of smaller refrigerators consts of (1) an exterior course of ⅞-in ngued and grooved lumber, (2) two ourses of water-proof insulatingaper and (3) a 3-in thickness of eet cork in two 1½-in courses, all oints being broken. To this insulaon is applied the interior lining.

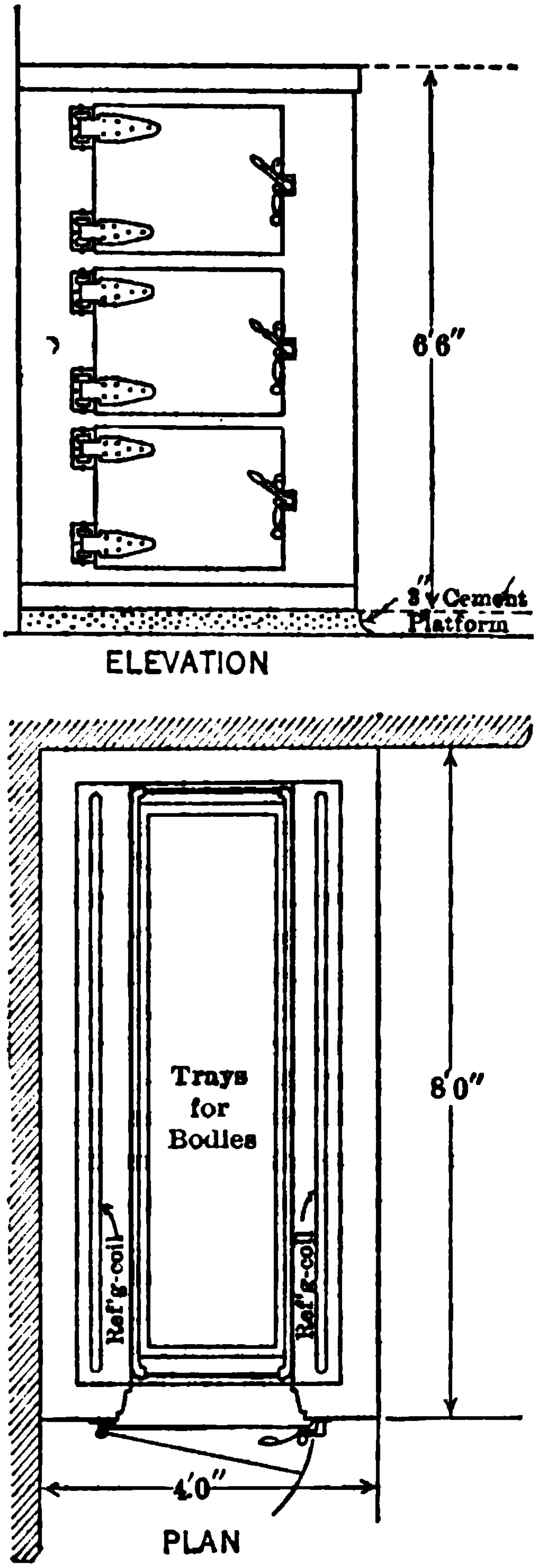


Fig. 4.* Mortuary-refrigerator

Mortuary-Refrigerators. Moruary-refrigerators should be cooled y mechanical refrigeration, the coils eing placed longitudinally on both ides of the mortuary-trays. Fig. 4 lustrates a mortuary-refrigerator for hree bodies. This may be used as a nit in designing mortuary-refrigerars of larger capacity, or the height ay be reduced to 5 ft and the odies placed in two instead of three orizontal tiers. Mortuary-refrigerators sometimes have both fronts finished nd equipped with doors so that bodies are accessible for identification or xamination from both fronts.

* The Jewett Refrigerator Company.

MECHANICAL REFRIGERATION *

A Brief Description of Methods in Common Use for Producing and Applying Refrigeration, with Special Reference to Small Plants

A British Thermal Unit, (Btu), is the quantity of heat required to raise the temperature of 1 lb of water 1° F. Heat used in this way, that is, to raise the temperature of water or other substance, is said to be present in that substance as SENSIBLE HEAT, or, in other words, heat, the presence of which we feel, or sense.

The Heat of Liquefaction, or so-called LATENT HEAT OF LIQUEFACTION of a mass of ice, is the amount of heat it will absorb in melting. One pound of ice at 32° F. will absorb 144 Btu in melting to water at 32° F. Heat coming into a cake of ice is thus absorbed in melting the ice and becomes what is known as LATENT HEAT, or heat absorbed without any rise in temperature. If the ice is at a lower temperature than 32° F., or if the water resulting from the melting rises above 32° F., additional heat will be absorbed as SENSIBLE HEAT.

The Specific Heat of a substance is the ratio of the quantity of heat required to raise the temperature of a certain weight of the substance one degree to that required to raise the same weight of water from 62° to 63° F.

The Heat of Vaporization of water or of any other liquid is the amount of heat it will absorb in vaporizing, in evaporating from a liquid to a gas, or will give out in returning from the gaseous to the liquid state.

Transfer of Heat occurs in three ways: (1) by convection, (2) by radiation and (3) by conduction. For instance, if particles of air in a refrigerator adjacent to a source of heat become warmed they circulate and distribute the heat by CONVECTION through the refrigerator-box. Heat will pass from a warm substance, as from the filament of an incandescent lamp, out into the box by RADIATION. Heat will enter the box through the walls by CONDUCTION.

Heat-Transmission. When the temperatures on opposite sides of any surface, as for instance, a wall, are unequal, heat will pass by conduction through the material from the warmer to the cooler side. The rate of this movement is the RATE OF HEAT-TRANSMISSION and is stated in terms of the quantity of heat called (Btu) which will pass through 1 sq ft of surface in 24 hours, per degree temperature-difference between the two sides of the wall.

Some Advantages Claimed for Mechanical Refrigeration.

(1) Lower temperatures can be obtained with refrigerating-machines than with ice.
(2) The inconvenience of handling ice is avoided.
(3) There is no accumulation of slime in the refrigerators as from the melting of even the best ice.
(4) Refrigerators cooled mechanically are dryer than ice-cooled boxes because the moisture is frozen out of the air and deposited on the cooling surfaces.
(5) There is generally a better air-circulation, resulting in a more uniform temperature and dryer atmosphere throughout the compartment.
(6) With proper design of refrigerator and refrigerating-machine any desired temperature can be obtained.
(7) Refrigeration produced mechanically is often cheaper than refrigeration produced by melting ice. (See page 1695.)

* Compiled and adapted, by permission, from data included in a paper by R. F. Mas... See, also, Refrigerators, pages 1679 to 1683.

eration of Refrigerating-Machines. In almost all methods of producing advantage is taken of the fact that when a liquid evaporates it usually cools itself and its surroundings, and changes into a gas or vapor. There are al liquids which are easily made to evaporate and produce this cooling , and were it not for their cost, refrigeration could be very simply produced pplying a steady stream of the liquid and allowing the vapor or gas evapo- l to escape into the atmosphere. A refrigerating-machine is practically an ratus for saving this gas which has evaporated and returning it to its liquid to be used over again. In this process of recovery and condensation the gives out the heat which it has previously absorbed in evaporating. This is carried away by flowing water, which, in absorbing the heat, rises in erature.

ypes of Refrigerating-Machines. In the (1) COMPRESSION-TYPE of re- rating-machines the recovery of the gas is effected by drawing it away from oint where it has been evaporated and pumping it under increased pressure a chamber where it gives out its heat to the water-cooled walls of the nber and returns to the liquid state ready to be used over again. In the ABSORPTION-TYPE of refrigerating-machines ammonia is generally used and recovery of the gas is effected by bringing it into contact with water with h it unites chemically. The solution thus formed is pumped into another nber, and heat is applied to drive off the ammonia-gas which is then condensed r high pressure. It is now ready to be reevaporated and reproduce its cool- effect. In all cases of large units, and in all cases of either large or small s where exhaust-steam is available in sufficient quantities, absorption re- rating-machines are very economical.

quids Used in Refrigerating-Machines. A number of liquids have been l in refrigerating-machines, the ones commonly employed being (1) AM- IA, (2) CARBON DIOXIDE and (3) SULPHUR DIOXIDE. Various practical iderations determine which is to be used in any particular design of machine. h (1) AMMONIA the advantage is the lower working pressures, from 15 to lb per sq in, which are easy to deal with. An advantage over carbon dioxide at leaks are very easily located. Ammonia-fumes, however, are offensive sometimes dangerous in case of a break. With (2) CARBON DIOXIDE the antage is in its inoffensive odor. Its disadvantages are the high pressure at h it works, from 300 to 1 200 lb per sq in, the relative difficulty of holding e pressures and of finding small leaks, owing to its slight odor and chemical tivity. With (3) SULPHUR DIOXIDE the advantage is its comparatively low king pressure, which is not above 75 lb per sq in. Its great disadvantage is : with moisture it forms an acid which rapidly corrodes the apparatus. At time this disadvantage was fatal, since with the old-type machines, air and sture were constantly being drawn into the system more or less rapidly and ed with the sulphur dioxide. This difficulty has recently been overcome in e modern types of machines * in which the refrigerant is hermetically sealed he machine and chemical action, therefore, prevented.

ating of Refrigerating-Machines. A 1-TON REFRIGERATING-MACHINE is a hine which, if operated for 24 hours, will absorb the amount of heat which n of ice would absorb in melting. If the machine is operated a shorter time day, a less amount of heat will of course be absorbed, and in order to main- the temperature during the period when the machine is not running, some

The Audiffren Refrigerating-Machine, a small machine intended for domestic uses manufactured by Johns-Manville, Inc., New York. There are many other reliable s making refrigerating-machines of other distinct types, and the architect should carefully into the merits and claims of each when called upon to specify them.

means must be adopted for storing cold. (See paragraph below.) Refrigera machines are sometimes rated in terms of ICE-MAKING CAPACITY, that is, in of the amount of ice the machine will make in 24 hours. This is always less the refrigerating capacity because some refrigerating effect is required to the water down to 32° F. before the freezing can begin, and the ice is us cooled several degrees below 32° F., which requires a still greater capacity. Th is also some flow of heat into the apparatus. These elements vary consider so that from some points of view ICE-MAKING CAPACITY might be considered unsatisfactory method of rating some refrigerating-machines.

Applying the Cold. According to one classification there are three com systems of applying the cold. These are, (1) the DIRECT-EXPANSION SYS (2) the BRINE-SYSTEM and (3) the COLD-AIR SYSTEM.

(1) In the DIRECT-EXPANSION SYSTEM the refrigerant is evaporated in a of pipe placed directly in the room to be cooled.

(2) In the BRINE-SYSTEM the refrigerant is used to cool brine, which is th circulated through coils of pipe in the room to be cooled.

(3) In the COLD-AIR SYSTEM a current of air is chilled by passing it over of pipe cooled directly by the evaporating refrigerant, or by brine, or passing it through a spray of cold brine; and this chilled air is then passed i the room and circulated back to the cooling-coils, the whole operation be repeated indefinitely.

All of these systems have their advantages and disadvantages. While t brine-system is a little more expensive to operate in large plants, the temp ature is more easily controlled than with the direct-expansion system, and practice in small plants it is found as economical in operation in spite of i theoretical disadvantage. Furthermore, in case of any breakdown in the m chine, the temperature can be held for a time by circulating the brine until becomes too warm to be of use, whereas with direct expansion the temperat will begin to rise immediately upon the stopping of the machine. The cold system is not as applicable where any drying of the goods stored would be har ful and there is some risk of carrying fire in the air-passages. It is much us nevertheless, for such service as chocolate-dipping rooms, ice-cream harden fur-storages, etc.

Storage of Cold. When temperatures are to be maintained while the refri erating-machine is shut down, COLD must be STORED. In the brine-system t is effected by cooling a comparatively large body of brine which warms slo as it is circulated. Where the brine-circulating pump as well as the machi must be stopped, so-called PRESSURE-TANKS may be placed in the pipi system in the room being cooled; the mass of brine in these tanks absorbs t heat and helps to maintain an approximately even temperature. Where th direct-expansion system is used, a part of the cooling-coils may be immersed i a tank of brine placed in the room and the remainder of the coils arranged f the direct cooling of the room. In some places the spaces available will n permit the use of brine-storage tanks. In cases of this kind smaller tanks m be used and filled with water, or a weak brine which will freeze at a tempera ture a little below 32° F. Since 1 lb of ice in melting will absorb 144 Btu an 1 lb of brine rising in temperature, say 20°, will absorb only from 14 to 16 Bt the saving of space is apparent. It must be absolutely certain that the refri erant reaches the tank first at the bottom and that the air to be cooled reach it first at the top so that the ice in forming shall not bulge or burst the tan If the congealing mass were to freeze from the top down the tank would b strained and finally leak, because of the expansion of the ice in freezing. A other fact to be considered is that where water, only, is frozen, a resulting hi

mperature may be obtained in the refrigerator, since the brine must be armer than the ice in order to melt it, and the refrigerator just that much armer, or warmer than an ice-cooled box. In calculating the proper sizes of nks for storing brine, it should be remembered that, usually, the period during hich the machine is shut down coincides with the period during which the mand for refrigeration in the box is the least. The amount of heat to be sorbed is usually only that entering through the insulation, as the doors are ut and no food is put in or removed.

Description of Refrigerating-Machines. As explained in the preceding ragraphs refrigerating-machines may be divided generally into two classes,) the COMPRESSION-TYPE and (2) the ABSORPTION-TYPE.

(1) **The Compression-Type of Refrigerating-Machines** may be subdivided as llows:

(*a*) The open type of machine, which is made both vertical and horizontal, d both single and double-acting, that is, compressing the gas at one end or both ends of the cylinder. (*b*) The partially enclosed type of machine, in hich all the moving parts of the compressor proper are enclosed within the frame the compressor, except the fly-wheel and the main shaft which enters the ame of the machine through a stuffing-box. Such valves, also, as are required the system are exposed. (*c*) The wholly enclosed type of machine,* in which l of the working parts are enclosed in a hermetically sealed container.

(*a*) One advantage of the open type of machine is that any lack of adjustment e to wear can be readily corrected; so that, with proper attention, it gives cellent results. For large installations this is considered by many to be a ost efficient type of machine.

(*b*) The enclosed type of machine resulted from the effort to reduce the amount attention required by the open machine, to cheapen its construction and to duce the possibility of trouble from inexpert tampering. An objection to achines of this type is that when adjustments have to be made the working rts are relatively inaccessible.

(*c*) With the wholly enclosed type of machine it is claimed that the loss of e refrigerant is prevented by the hermetical sealing of the apparatus, and that e working parts, being completely enclosed, are protected from deterioration e to outside causes or tampering.

(2) **The Absorption-Type of Refrigerating-Machines** are of two kinds, differ-g principally in the proportioning of the parts. In the one machine high-pres-re steam is used; in the other the proportions are such that low-pressure exhaust-steam may be used. Where exhaust-steam is available machines this type are found to be very economical, and this is true, also, for all large its whether or not exhaust-steam is used. Full descriptions of these machines ith detailed plans and layouts may be obtained from the various manufacturers.

Calculations for the Capacity of a Refrigerating-Machine. Heat enters e refrigerated compartments, (1) through the walls, (2) with warm goods, (3) the interchange of the outside air when doors are opened and by air-leaks, nce the cooled air is the heavier and immediately flows out when a door is ened, (4) from lights or from the heat of the bodies of workers, and (5) om any change of state occurring in the goods, such as freezing, fermenting, c. In large rooms these various sources of heat should be analyzed sepa-tely. In small refrigerators, as in hotels, kitchens, dwellings, etc., a rough le, quite as accurate as a more elaborate analysis, allows a certain number of tu per cubic foot of refrigerated space per 24 hours. This amount varies

* Referred to on page 1685.

with the character and location of the box, the nature of its insulation, the temperatures desired and so on. It will be seen that the insulation, while of great importance, is not by any means the only important factor in this class of boxes. For domestic refrigerators in which a temperature of from 35 to 50° F. is maintained, 300 Btu per cu ft of refrigerator per 24 hours should be allowed. For boxes in hotel or restaurant-kitchens, 600 Btu, or even 900 Btu in extreme cases and where low temperatures are required, should be allowed. For butchers' coolers or large storage-boxes in hotels, etc., from 200 to 250 Btu per cu ft per 24 hours should be allowed. A check on the above figures for the large type of box is the following: * "When the exact conditions under which cold-storage rooms are to be operated are known, namely, the size and shape of the rooms, the quality of the insulation, the kind and quantity of goods to be handled per day and the temperatures at which they are received and at which they are to be held, the amount of refrigeration required can be estimated very closely by the following rule: (1) Calculate the exact area of exposed surface in the walls, floor and ceiling of the room in square feet, multiply the total number of square feet by the number given in the table for the required temperature and divide the product by 288 000. (2) Multiply the amount of goods in pounds, to be stored per day by the number of degrees of heat to be extracted by the specific heat of the goods, and divide by 288 000. This will give the amount of refrigeration, in tons per day, necessary to maintain the temperature required for the goods. (3) Add these two amounts together. The total will be the amount of refrigeration, in tons per day, required to maintain the temperature required for the goods and for the room. (4) If the goods are to be frozen, the latent heat of freezing should be added to the number of Btu to be extracted."

For rooms containing less than 1 000 cu ft	
If maintained at 0° F. multiply the exposed surface by	1 775
If maintained at 5° F. multiply the exposed surface by	710
If maintained at 10° F. multiply the exposed surface by	535
If maintained at 20° F. multiply the exposed surface by	355
If maintained at 32° F. multiply the exposed surface by	265
If maintained at 36° F. multiply the exposed surface by	180
For rooms containing from 1 000 to 10 000 cu ft	
If maintained at 0° F. multiply the exposed surface by	1 250
If maintained at 5° F. multiply the exposed surface by	600
If maintained at 10° F. multiply the exposed surface by	300
If maintained at 20° F. multiply the exposed surface by	190
If maintained at 32° F. multiply the exposed surface by	160
If maintained at 36° F. multiply the exposed surface by	125
For rooms containing more than 10 000 cu ft	
If maintained at 0° F. multiply the exposed surface by	1 100
If maintained at 5° F. multiply the exposed surface by	550
If maintained at 10° F. multiply the exposed surface by	275
If maintained at 20° F. multiply the exposed surface by	180
If maintained at 32° F. multiply the exposed surface by	140
If maintained at 36° F. multiply the exposed surface by	110

* Taken from Levey's Refrigeration Memoranda, page 41.

With small machines it is necessary to allow a greater capacity of machine : a given size of box than with large machines, since, with the latter, one can vays throw a large part of the machine-capacity to any given box where ɔcial need may exist; whereas to do this with the small machine would almost tainly rob some other box, if indeed there happened to be another box. It is ver possible to determine with mathematical certainty exactly how much rigeration is required for a given case. It is best to allow for this fact and be sure the machine is amply large. Where an existing ice-cooled box is to cooled mechanically one check upon the size of the machine required is the ount of ice used. This check is more apt than any other, however, to lead erroneous conclusions unless the figures are properly analyzed.

Another Method of Determining the Capacity of a Refrigerating-Machine. e following is a method that gives good results, except that allowance may be de in the larger boxes and where brine-storage tanks are provided in the box the steadying effect of the mass of cold brine:

(1) The ice-consumption for the hottest month of the year should be deter-ned. This will give the average ice-consumption for that month.

(2) The average temperature that is maintained in the box with ice should n be accurately determined. This will usually be from 55 to 65° F. It will nmonly be stated to be anywhere from 40 to 45° F., but these temperatures : seldom obtained. Even if they are, with a full ice-chamber and the box sed for long periods the average will be above these figures. Unless, there-e, there is positive assurance to the contrary, from 55 to 60° F. should be sidered the average temperatures.

(3) A calculation should then be made of the heat-inflow through the insula-n, with a temperature of 55° F. in the box and with the average summer nperature outside. The difference between the heat-inflow through the ulation and the total heat actually absorbed by the melting of the ice is the ount entering the box from other sources than through the insulation. This ess of heat ordinarily occurs during the hours of daytime only, that is, when : box is being opened, since at night the box will remain closed. A machine sufficient capacity to produce the temperature actually obtained with ice st, therefore, be of larger rated capacity than that indicated by the actual -consumption; and how much larger it should be can be determined by this thod.

(4) A further fact which it is claimed should be taken into account in deter-ning the proper size of a machine is that temperatures obtainable with ice : often unsatisfactory. If they were always satisfactory one reason for put-g in cooling-machinery would be done away with. Where 55° F. is obtained h ice, from 35 to 45° F. will be required with mechanical cooling and the chine-size must be further increased in the ratio of the temperature-differ-es between average summer temperatures and 35° F., and average summer nperatures and 55° F.

(5) The cooling-machine if installed in accordance with these figures would ndle average-weather conditions but would not be adequate for extreme -weather conditions, the most important conditions to be met by cooling-chinery. It is necessary, therefore, to further increase the size of the machine the ratio of the difference in temperature between maximum summer tem-ature and 35° F., and average summer temperature and 35° F.

(6) A further allowance should be considered, namely, the fact that in many es, for one reason or another, it is not possible, or else not desirable, to oper-the machine except during certain periods of the day, and the machine-size st be increased as much as may be required to take care of these conditions.

(7) If the machine is not placed directly at the box to be cooled, allowance must be made for the heat-inflow into the insulated brine-mains. The amount of heat entering from this source is often of considerable importance, particularly with small machines. The table below gives heat transmissions for cork pipe-covering and some other materials.

Water and Milk-Cooling. Mechanical refrigeration as applied to cooling water and milk differs in one respect from other classes of refrigerating-work. A relatively intense quantity of cooling effect is called for in a brief interval of time. For instance, in a drinking-water system the heaviest requirement may come at the noon-hour. In a bakery, also, the demand for chilled water will be intermittent, a large quantity of water being required for the dough mixing. In dairy-work the milk must be cooled very rapidly to check the development of bacteria which grow with incredible rapidity within the temperature-range of from 110 to 50° F. To install a large enough refrigerating machine to produce the required cooling effect as it is needed would in most cases call for a very large machine. This is overcome by using a small machine and allowing it to operate for a longer time, say throughout the day, storing the refrigerating effect produced by cooling a large body of brine or melting the ice as rapidly as may be required. For instance, if 50 cans of milk, of 40 qts each, are to be cooled from a temperature of from, say, 75 to 35° F., in 1 hour, the refrigeration required will be 50 cans times 40 qts times 2 lb per qt times (75° F. − 35° F.), which equals 320 000 Btu. Milk is treated in the calculation as having the same specific heat as water, since water is so large a percentage of its total weight. This amount of refrigeration produced by a machine running 12 hours per day would require the machine to absorb 320 000 Btu divided by 12, or 26 000 Btu per hour. The quantity of brine necessary to store the cooling effect may be calculated closely enough for practical purposes by using the following approximate figures. The specific heat of brine is 0.75. The weight of the brine is 9 lb per gallon. The permissible temperature-range of the brine depends upon the conditions and may be from, say, 30 to 15° F., or lower. In other words, the temperature to which the brine can be permitted to rise is limited to the temperature it must produce in the room or in the substance being cooled, and the temperature to which the brine can be cooled in storing cold is limited by the decrease in economy of the refrigerating-machine at the low temperatures.

The Value of Good Insulation. (See, also, Insulation, page 1683.) The importance of good insulation cannot be too strongly emphasized. A cold-storage room or refrigerator and its contents may be cooled by ice or mechanical means, but unless the walls are adequately insulated, the demand caused by the inflow of heat through the poor insulation may be more than the ice supply or refrigerating-machine can meet to maintain the required temperature. The almost universal standard of insulation for cold-storage rooms is 4-in thickness of pure-cork sheet. The following table shows the heat transmitted through 1 in in thickness of each of the substances, per square foot of exposed surface per degree difference in temperature per 24 hours.

Pure-cork sheets	6.4 Btu
Hair-felt	7.3 Btu
Impregnated cork boards	8.5 Btu
Rock-wool blocks	8.0 Btu
Waterproofing lith-blocks	8.5 Btu
Spruce, clear and dry	16.0 Btu
White oak	26.0 Btu

Design of Refrigerators. Disposition of Cooling-Surfaces. (See, also, bject of Refrigerators, page 1679.) No attempt need be made to describe all the many arrangements of refrigerated compartments that are to be found in rvice. The intention is to point out some of the more important things to considered in determining upon the design of a box. It is desirable in a rigerator to produce not only a low temperature, but a relatively dry atmosere.

Cooling-Surface and Temperature. Securing the low temperature is merely question of supplying sufficient cooling-surface to produce the desired results th the temperature available in the refrigerant. The amount of surface quired is influenced by the arrangement of the box, that is, whether or not e air passes freely or sluggishly over the surface, whether the cooling-surface placed on the ceiling or walls of the compartment or in a loft and, if the latter rangement is used, whether or not the air-passages are of proper size and the culation between the loft and the compartment sufficient.

Dryness of Atmosphere and Temperature. To secure a box of satisfactory yness it is necessary to have a relatively low temperature in the refrigerant. e air which passes over the cooling-surfaces is practically in a saturated conion when it leaves them. If it is to be dry at the temperature required in e box, it must have been, necessarily, cooled well below the box-temperature. r instance, in a box, the temperature of which is maintained at 35° F., the ne should be run at a temperature of from about 20° to 25° F. It is further sirable to so locate the cooling-surface that frost in melting will pass out of e box quickly and not remain to be reabsorbed by the air in the box.

Arrangements of Cooling-Surfaces. There are several common arrangements cooling-surfaces in refrigerators. Sometimes the coils are arranged overhead, t directly in the compartment to be cooled. This is one of the efficient ways which a cooling-surface can be arranged, so far as the cooling effect alone is ncerned. It is not, in general, a good arrangement, however, since frost lting from the coils drips on the goods. In another arrangement the coolingfaces are on the wall. This is preferable to the ceiling-arrangement, as far the dripping is concerned. The objection to it is that goods placed close to e walls are apt to be overchilled, while goods nearer the center of the comrtment are not cooled quickly enough. It also wastes floor-space, because cking goods close to the coils is not practicable on account of possible overlling and also on account of the liability of retarding the air-circulation. The ll-arrangement for cooling-surfaces is, nevertheless, often the most practica- method. Another method involves a modified form of wall-coil arrangement which a brine-storage tank is used to assist in maintaining the temperature en the machine is shut down. A further modification is often introduced, which a partition or baffle-plate is used in front of the coils. The best types box-arrangement are those in which the cooling-surface is separated from the rage-space and is so arranged as to secure an active circulation of the air r the coils and through the compartments. In all of these plans the one quirement calling for the greatest care is that the air-passages shall be as ect as possible and of ample size. The force causing the air to circulate, nely, the difference in weight due to differences in temperature and density tween the column of air in the coil-compartment and that in the storagempartment, is so extremely small that any slight interference is a serious tter. An extra turn in the passage or a slight reduction in the size of the ssage will produce a marked effect. A good rule to follow is to make the ssage as large as it can be made without allowing any drip to reach the rage-compartment. This will work out in many cases to show a ratio of

1 to 8 or 9 between the area of the passage and the floor-area of the com-ment; but even 1 to 6 is just that much better if it can be secured. The mat of proportioning the size of the air-passages is of much less importance wh the air is circulated by fans. Forced circulation is not usual, however, ex in large storage-refrigerators, and no attempt will be made here to consid One precaution that must be taken in arranging the cooling-surface, especi in small and frequently opened boxes, is the avoidance of any undue cooling walls or ceilings that are exposed to currents of warm air when the do opened. Moisture from the incoming air deposits on these surfaces and ca the offensive so-called SWEATING of the box. This is most often seen on t storage-compartment side of uninsulated coil-compartment floors or partiti and also occurs on walls or ceilings where the cooling-pipes are set very close these surfaces. The obvious and effective cure is to insulate the partiti between coil-compartments and storage-compartments and keep cooling-s faces well away from walls or ceilings, from 3 to 8 in, depending upon the te perature of the brine.

Incidental Notes on Refrigerators. Drawers. In restaurant-kitch and elsewhere it is sometimes convenient to have a box fitted with a numb of refrigerated drawers. The heat-leakage through the many joints, throu slides which are invariably only partially closed, and through the poor insul tion of the drawers, is very great. Where it is at all possible to do so, it best to arrange an insulated door covering the entire drawer-space.

Anterooms. In storage-rooms of medium to large size the air-interchan due to opening doors is reduced to a minimum by arranging an anteroom entry which, after it is entered, has its outer door closed before the door to t storage-room proper is opened. Where two rooms are side by side, it is oft possible to reduce the interchange of air by treating the one room as an an room of the other, having but one door to the outside air.

Doors. Special note should be made as to the design of doors for refrigera rooms or boxes. There is a common idea that a refrigerator-door should beveled. As a matter of fact no more certain means of ensuring air-leak could be devised. A perfectly fitted beveled door, hung accurately in pla could perhaps be made tight in the beginning. This door in service at on begins to sag, since a refrigerator-door is always heavy. It immediately b comes impossible to force it to a tight seat and continuous leakage of air begi A refrigerator-compartment door is most readily made tight by having a f surface on the door come up against a corresponding surface on the frame, wi a soft gasket of some kind between them. There are several well made r frigerator-doors on the market at prices low enough to make it doubtful econom to attempt the home-made article.

Arrangement of Brine-Mains. In laying out mains to carry brine from th refrigerating-machine to the refrigerator, there are a few simple points to b cared for. For the convenience of the pipe-covering man, the flow and retu lines should be placed far enough apart so that he can get his covering on each pipe without cutting it to pieces, or else they should come close togeth so as to be covered together. A common difficulty experienced in brine-sy tems of refrigeration, where the cooling-coils in several compartments are fe from the same main, is that when the adjustment of the valve controlling th flow of brine through one coil is changed, it upsets the adjustment of the whol system. This is due to too small mains or too small a pump, or both. A similar action is observed when the opening of a faucet on a water-pipe chec the flow from other open faucets on the line. The ideal cross-section area o

brine-mains is as nearly as possible equal to the combined cross-section of the coils which they serve at any one time. Even with this proportion, ever, it is not possible to absolutely ensure that the lower coils will not rob upper ones, or even drain them completely in some systems of piping. A effective, even if somewhat expensive method of overcoming this diffi-, is by the addition of a third main. In this arrangement it is not possible ne coil to rob another to the point of draining it.

alculations for the Necessary Amount of Cooling-Surfaces. No and fast rule can be given regarding the proper amount of cooling-surface compartments of various sizes, since the design and arrangement of the ng-surface and the freedom with which the air circulates over it greatly t the amount required. As a general guide, however, and where the con-ns are such as to permit a good circulation of the air, the following formula give good results. It will be understood, of course, that the refrigeration ired in the given room has been determined as previously indicated. cooling-surface required, in square feet, per ton of refrigeration equals $0/(T - t)$ in which T is the temperature desired in the compartment, and t average temperature of the brine.

Approved Cold-Storage Temperatures

Articles stored	Degrees Fahrenheit
ef	36 to 40
mb and mutton	32 to 36
gs	29 to 32
al	34 to 36
eats, in pickle or brine	35 to 40
tter, must be kept separate from other goods	0 to 38
gs	29 to 32
eese	32 to 34
rd	38 to 40
ultry, to freeze	5 to 10
ultry, when frozen	25 to 28
me, to freeze	5 to 10
me, when frozen	25 to 28
h, retail fish-counters should be cooled with ice rather than mechanically	25 to 28
sters	33 to 45
er	33 to 42
ines	40 to 45
der	30 to 40
uits	33 to 36
getables	34 to 40
nned goods	38 to 40
our and meal	40
rs	25 to 32
ine for ice-cream freezing	5 to 10
e-cream, air-hardening	5
e-cream, serving-temperature	14 to 16

e-Making. If the following facts of physics are kept in mind in consider- methods of making ice the results obtainable may be understood or pre-ed:

(1) Chemically pure water will freeze solid and clear.
(2) Water containing impurities in solution tends in freezing to force th impurities out of solution. The slower the process of freezing t more completely is the purification effected.
(3) Ice forming in still water sends out long slender crystals which incre in number and size, forming a meshwork that gradually becomes solid mass.
(4) Agitation of water during freezing aids in the separation of impuri and therefore in forming solid, clear ice.
(5) Practically all natural waters contain more or less organic or inorga material in solution and invariably contain air in solution. Th substances are, therefore, frozen out of solution and tend to cause t ice formed to be opaque, the lighter substances tending to rise a collect near the surface, and the heavier ones tending to sink.
(6) The rate of freezing of ice decreases as the thickness already form increases, so that the time required to freeze increases as the squa of the thickness to be frozen. In the formation of natural ice t freezing is from the top down and impurities frozen out of solution fi This and the motion of the water, especially in quiet running strea tends to make naturally frozen ice transparent. American manuf turers of ice have always tried to duplicate this clearness.

Methods of Ice-Making. The method first adopted in this country was t one in which DISTILLED WATER was used. From a sanitary point of view su ice would be theoretically ideal. Practical difficulties make it almost imp sible to secure pure ice in this way. Some of these difficulties are:

(1) Removal of oil from the distilled water, this oil being picked up as t steam passes through the cylinder of the engine. It is difficult remove organic oil which is present in the lubricant.
(2) Assurance that the filters are in proper shape, an assurance often imp sible to obtain since this apparatus is ordinarily used the season throu without overhauling.
(3) Possibility of contamination in the storage-tank where the distilled wat is held and usually PRECOOLED to as near 32° F. as possible, befo passing to the freezing-cans, thus saving time in the freezing proc in the tank.
(4) Possible contamination from handling the cans and the wooden cov over them. These covers form the top of the freezing-tank in whi the cans of water are immersed in cold brine for freezing and a tramped over by the ice-harvester with the consequent possibility dirt getting into the cans.

A second system of ice-making in common use in this country is the PLA SYSTEM. In this process the ice is formed on vertical steel plates. Natu or raw water is used and the bath is agitated by various methods. The r sulting ice is very clear and dense. In this system when the ice is formed the desired thickness, usually about 12 in, it is loosened from the freezi plate by various thawing-arrangements in different forms of the apparatu The ice-plates, often 9 by 16 ft by 12 in in thickness, are lifted from the tan by overhead cranes and carried to a table where they are cut to commerci sizes. While the plate process is usually very slow on account of the f that the freezing is from one side only, it is largely used and lends itself to gr economy in steam-consumption, whereas in the old-style distilled-water i making plant the amount of steam required to make the ice was more than t

nomical engine would use and it was not possible to obtain fuel-economy. e modified form of this system, now coming into considerable favor, is anged so that STATIONARY CANS are filled with raw water and kept agitated compressed air bubbling up through it. When the freezing has progressed newhat the remaining water is drawn off and replaced by fresh water, thus noving the greater part of the impurities that have been frozen out of solu-n. Various other modifications of these two systems of ice-making have been l are being developed. All of them depend, however, upon the series of ysical facts stated in the preceding paragraphs, and the results may be lyzed by reference to them.

Relative Economy of Producing Refrigeration Mechanically and by . (1) In determining the cost of REFRIGERATION BY ICE, account must taken not only of the cost of the ice but of melting, of the uncertain ice-vest, of the amount of ice left over at the end of the season and of that zen together in the storage and, therefore, practically useless. Regarding melting, it may run anywhere up to 50% of the total ice-harvest. The antity left over at the end of the season is, of course, so variable that it is possible to estimate it, this being purely a matter of chance. In many es, however, it is a very large item. The loss by the ice freezing together the storage can be reduced to a very small amount where the ice is properly ked with distance-strips between the ice-cakes. Proper packing is much re readily carried out, however, where artificial ice is stored than where ural ice is held, and a mechanically cooled ice-storage is less subject to this iculty, since the temperature is, of course, constantly held below the melting-nt of ice. (2) The total cost of REFRIGERATION PRODUCED MECHANICALLY ludes the cost of power, water, oil, refrigerant (usually ammonia), labor l attendance, and interest and depreciation on the investment. The figures these items vary between wide limits. The following figures, however, will of interest. Care should be taken in drawing conclusions from them as to t in prospective installations. These figures are from the annual cost of an -manufacturing company having a capacity of 1 500 tons per day in plants ging in size from 50 to 100 tons per day each.

Coal	40 cts per ton of ice produced.
Labor	50 cts per ton of ice produced.
Ammonia	10 cts per ton of ice produced.
Water	5 cts per ton of ice produced.
Waste, power, oil, etc	10 cts per ton of ice produced.
Total	$1.15 per ton of ice produced.

TOWER-CLOCKS *

Rule for Diameter of Dials. "To look well and show plainly, dials should 1 ft in diameter for every 10 ft of elevation and should set out flush with or se to the line of the building or tower." †

Dimensions of Some Large Clock-Faces. Colgate's Factory, Jersey ty, N. J. The diameter of the dial is 40 ft. The minute-hand is 20 ft long d 2 ft 11 in in extreme width, and the hour-hand is 15 ft long and 3 ft 10 in extreme width. The minute-hand weighs 640 lb and the hour-hand 500 lb. is is the largest clock in the world.

* For a description of the requirements of installation of tower-clocks, see page 154 "Churches and Chapels," by F. E. Kidder.
† Seth Thomas Clock Company, Thomaston, Conn.

Bromo-Seltzer Building, Baltimore, Md. The dials are 24 ft in di
The minute-hand is 12 ft 7 in and the hour-hand 9 ft 8 in from tip to tip
minute-hand weighs 175 lb, the hour-hand 145 lb.

Daniels-Fisher Building, Denver, Colo. The dials are 15 ft 6 in in dia
The minute-hand is 7 ft 10 in and the hour-hand 5 ft 7 in long.

Maryland Casualty Building, Baltimore, Md. The dials are 17 ft in d
eter. The minute-hand is 8 ft 4 in and the hour-hand 5 ft 11 in long.

Elgin Watch Company's Factory, Elgin, Ill. The dials are 14 ft 6 in in d
eter. The minute-hand is 7 ft 4 in and the hour-hand 5 ft 4 in long.

Tower-clock, Station of the Central Railroad of New Jersey, at Com
paw, N. J. The diameter of the single dial is 14 ft 3 in; the minute-ha
7 ft long and weighs 40 lb; the hour-hand is 5 ft long and weighs 28 lb.
motive power is furnished by a weight of 700 lb, hung from a ⅜-in steel ca

Four-dial clock, Produce Exchange Building, New York. The diamete
each dial is 12 ft 6 in.

Four-dial clock, Chronicle Tower,* San Francisco, Cal. The diameter of e
dial is 16 ft 6 in; length of minute-hands, 8 ft; length of hour-hands, 5 ft 6
The mechanism of the clock is 6 ft 1 in high and weighs 3 000 lb.

Pneumatic clock, City Hall and Court-House, Minneapolis, Minn. The di
are 23 ft 4 in in diameter.

LIBRARY BOOK-STACKS

The Stack-Work in General. The stack-room of a library is usually c
off by fire-proof doors from the rest of the building. The customary prac
among architects is to make the stack-work a separate contract and have t
general contractor turn the stack-room over to the stack-contractor with fi
ished floors, walls and ceilings. The stacks, made entirely of incombusti
materials, are then built as an independent structure.

Book-Ranges. The book-ranges are usually double-faced and are pl
in parallel rows with aisles between. The minimum aisle-width is about 2
4 in. Radial ranges waste space and are costly. Single-faced ranges a
relatively more expensive than double-faced ranges.

Tiers. All stacks are divided in their height into tiers by deck-floors
order that all shelves may be easily reached. The regular tier-height is 7
or 7 ft 6 in.

Deck-Floors. Deck-floors are composed of slabs of ¾-in rough pl
glass or 1¼-in white marble, supported on steel framework. A long, narr
opening or deck-slit is left between the edge of each deck-floor and the face
each range to allow proper ventilation of the stack-tiers. The net thickn
from top of deck to bottom of steel framework is from 3¼ to 3¾ in for ordina
spans. The deck-floors are carried by the shelf-supports.

Vertical Communication. Continuous flights of stairs of simple desi
and construction are placed at central points. Books are moved up and do
by means of dumb-waiters operated by hand, for short runs, or by electr
power controlled by push-buttons.

Shelf-Supports. The shelf-supports are made in various ways, diffe
with each manufacturer. In the best construction they extend the full wi
of the shelves so as to hold up the shelves and books without the use of
projecting brackets. They are made of sufficient strength to carry the co
bined loads of books, deck-floors and superimposed stack-tiers. They sh

* Destroyed in the earthquake and fire.

ovide for a uniform shelf-adjustment at intervals of about 1 in. Compactness is important. Open-work shelf-supports promote proper lighting and ventilation.

Shelves. In each tier of regular height there are usually six rows of adjustable shelves and one row of fixed shelves. Shelves are generally 8 or 10 in wide and 3 ft long. Other sizes are supplied if necessary. The adjustable shelves are made of solid plates of sheet steel or of parallel bars with spaces between. The fixed shelves are placed about 2 in above each floor-level. They are made of solid plates of steel to form dust-stops, fire-stops and water-stops between the tiers.

Finish. The adjustable shelves are always completely finished with baked enamel before delivery. The fixed parts, also, of the stack-construction may be finished at the shop with baked enamel, or preferably with air-drying enamel, after erection at the building, so as to permit repair.

Lighting. Electric-light wires are carried in metal conduits supported by the steel framework of the deck-floors. Lights of 16 candle-power are spaced about 6 ft apart in range-aisles and 12 ft apart in main aisles.

Heating. Indirect radiation is best for books. The lower tiers, only, of a stack should be heated, to prevent the upper tiers from becoming too warm.

Ventilation. Large stacks are usually ventilated artificially to prevent the entry of dust and outside air through open windows. In the Library of Congress, in Washington, D.C., fresh, filtered and tempered air is forced in at the bottom tier, finds its way up through the stack by means of the deck-slits and is drawn out at the top tier.

Weights. The shelves and shelf-supports * weigh from 7 to 10 lb per cu ft of book-range. Books weigh about 20 or 25 lb per cu ft of book-range. The steel deck-floor framing weighs from 4 to 6 lb per sq ft of gross area of deck-floor. Marble floor-slabs, 1¼ in thick, weigh about 20 lb per sq ft, and ¾-in rough, plate-glass slabs, about 10 lb per sq ft of net area.

Book-Capacities. Book-capacities per linear foot of shelf may be figured on the following basis: law-books, 5 volumes; reference books, 6 volumes; scientific books, 7 volumes; general literature, from 8 to 10 volumes. The average in the Library of Congress is 8½ volumes per linear foot. An ordinary stack-tier, 7 shelves high with double-faced ranges 16 in deep (or 8-in shelves) and aisles 32 in wide, with a reasonable allowance made for cross-aisles, stairways, etc., will contain about 22 volumes per sq ft of gross area.

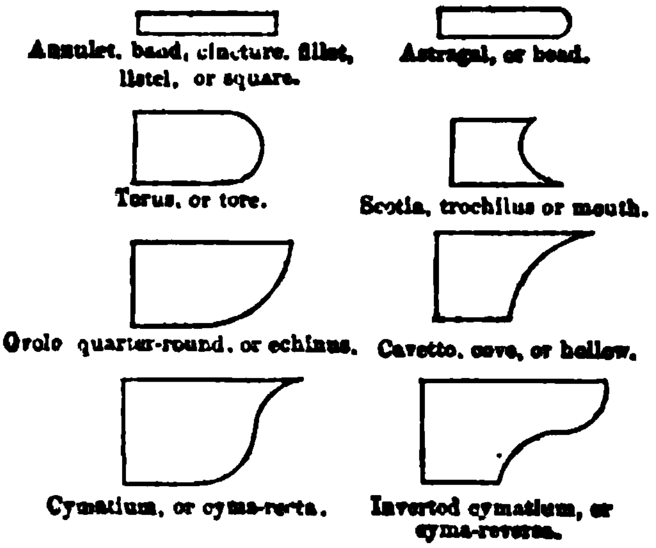
Cost. The cost in the United States of library-stacks of standard construction varies from 50 cts to $1 or more per linear foot of shelving. Economy is secured by following established standards while special designs increase the cost.

CLASSICAL MOLDINGS

Moldings are so called because they are of the same shape throughout their length as though the whole had been cast in the same mold or form. The regular moldings, as found in remains of classic architecture, are eight in number, as shown in the accompanying illustration, and are known by the following names: The last two are commonly called, also, OGEE MOLDINGS. Some of these names are derived thus: FILLET, from the French word FIL, a thread; ASTRAGAL, from ASTRAGALOS, a bone of the heel, or the curvature of the heel; BEAD, because the molding, when properly carved, resembles a string of beads; TORUS, or TORE,

* As made by Snead & Co., Jersey City, N. J.

the Greek for rope, which it resembles when on the base of a column; sco from SKOTIA, darkness, because of the strong shadow cast in its hollow, and wh is increased by the projection of the torus above it; OVOLO, from OVUM, an which this member resembles when carved, as in the Ionic capital; CAVE from CAVUS, hollow; CYMATIUM, fr KUMATON, a wave.



Characteristics of Moldin None of these moldings is peculiar t any one of the orders of architectu and although each has its appropri use, it is by no means confined to a certain position in an assemblage moldings. The use of the fillet an also of the astragal and torus, whi resemble ropes, is to bind the par The ovolo and cyma-reversa are stro at their upper extremities, and a therefore used to support projecti parts above them. The cyma-recta and cavetto, being weak at their upp extremities, are not used as supporters, but are placed uppermost to cover an shelter the upper parts. The scotia is introduced in the base of a column t separate the upper and lower torus, and to produce a pleasing variety an relief. The form of the bead and that of the torus are the same; the reas for giving distinct names to them is that the torus, in every order, is alwa considerably larger than the bead and is placed among the base-molding whereas the bead is never placed there, but on the capital or entablatu The torus, also, is seldom carved, whereas the bead is; and while the tor among the Greeks, was frequently elliptical in its form, the bead retains i circular shape. While the scotia is the reverse of the torus, the cavetto is th reverse of the ovolo, and the cyma-recta and cyma-reversa are combinatio of the ovolo and cavetto.

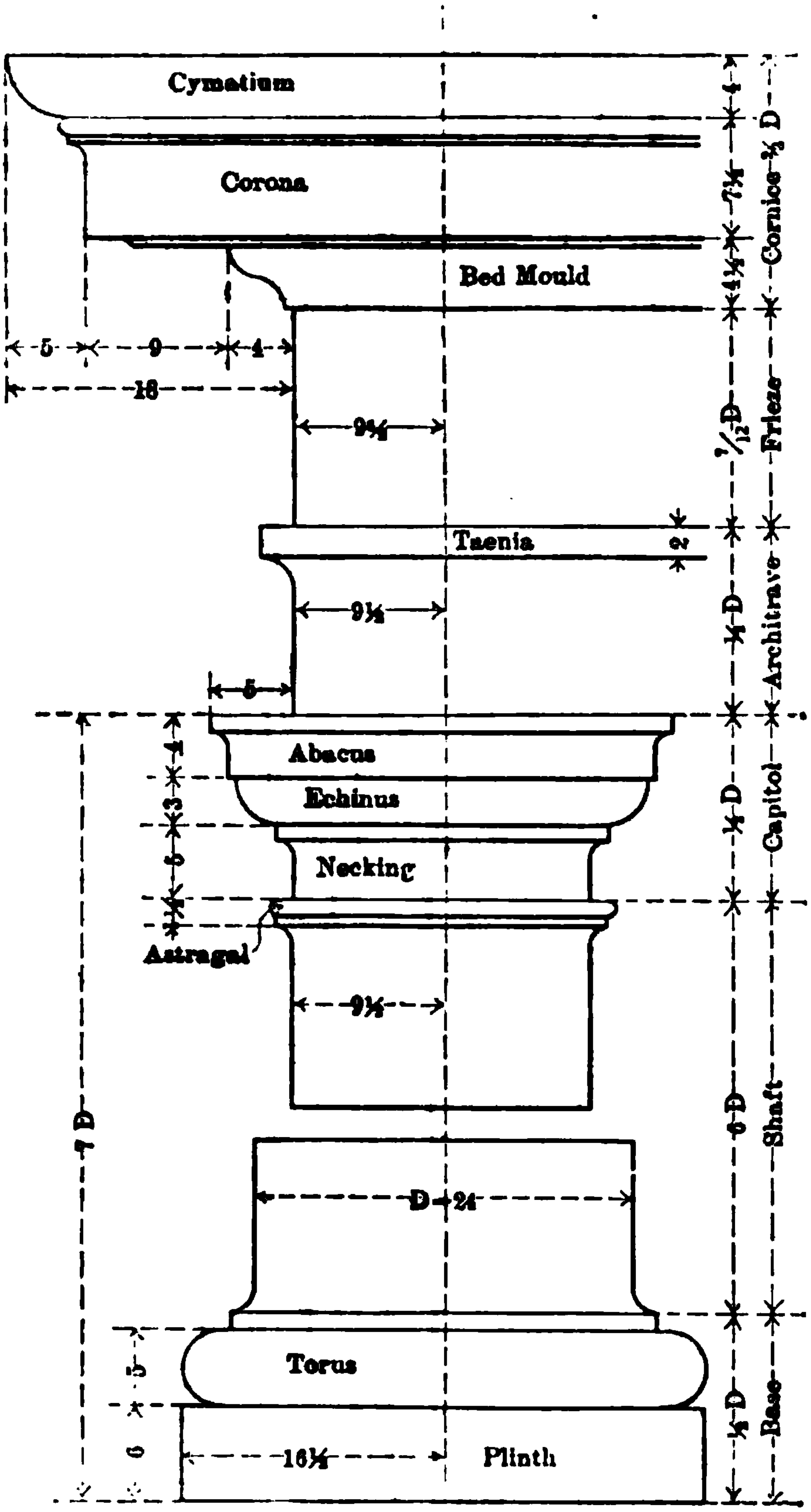
THE CLASSICAL ORDERS*

Origin of the Orders. "In the classical styles several varieties of colu and entablature are in use. These are called the ORDERS. Each order compris a COLUMN with a BASE, SHAFT and CAPITAL, with or without a PEDESTAL, with i BASE, DIE and CAP, and is crowned by an ENTABLATURE, consisting of ARCHITRAV FRIEZE and CORNICE. The entablature is generally about one-fourth as high a the column, and the pedestal one-third, more or less. Among the Greeks th forms used by the Doric race, which inhabited Greece itself and had colonies i Sicily and Italy, were much unlike those of the Ionic race, which inhabited th western coast of Asia Minor, and whose art was greatly influenced by that o Assyria and Persia. Besides the IONIC and DORIC styles, the Romans devise a third, which employed brackets, called MODILLIONS, in the cornice, and wa much more elaborate than either of them; this they called the CORINTHIA They used also a simple Doric called the TUSCAN, and a cross between th Corinthian and Ionic called the COMPOSITE. These are the FIVE ORDERS. Th

* The paragraphs in quotation-marks are taken from The American Vignola by Pr fessor W. R. Ware, by permission of the owners of the copyright, the International Tex book Company, Scranton, Pa., proprietors of the International Correspondence Schoo The engravings were made especially for this book, and correspond with the origi drawings prepared by Giacomo Barozzi da Vignola.

cient examples vary much among themselves and differ in different places, d in modern times still further varieties are found in Italy, Spain, France, ermany and England. The best known and most admired forms for the orders e those worked out by Giacomo Barozzi da Vignola in the sixteenth century om the study of ancient amples."

The Tuscan Order. The distinguishing aracteristic of the Tus- AN ORDER (Fig. 1) is mplicity. Any forms pedestal, column and tablature that show ut few moldings, and ose plain, are con- dered to be TUSCAN."



Dimensions are in 24ths of Diameter.

Fig. 1. The Tuscan Order

The Doric Order. The distinguishing aracteristics of the ORIC ORDER are fea- ires in the FRIEZE and the BED-MOLD above called TRIGLYPHS and UTULES, which are sup- osed to be derived from ie ends of beams and afters in a primitive ooden construction ith large beams. Un- er each triglyph, and eneath the TÆNIA which rowns the architrave, is little fillet called the EGULA. Under the egula are six long drops, alled GUTTÆ, which are ometimes conical, some- imes pyramidal. There re also either eighteen r thirty-six short cylin- rical guttæ under the ffit of each mutule. he guttæ are supposed o represent the heads of ooden pins, or treenails. wo different Doric cor- ices are in use, the MUTULARY with bracket and the DENTICULATED with dentils, he principal difference being in the BED-MOLD." The order shown in Fig. 2 as the denticulated cornice.

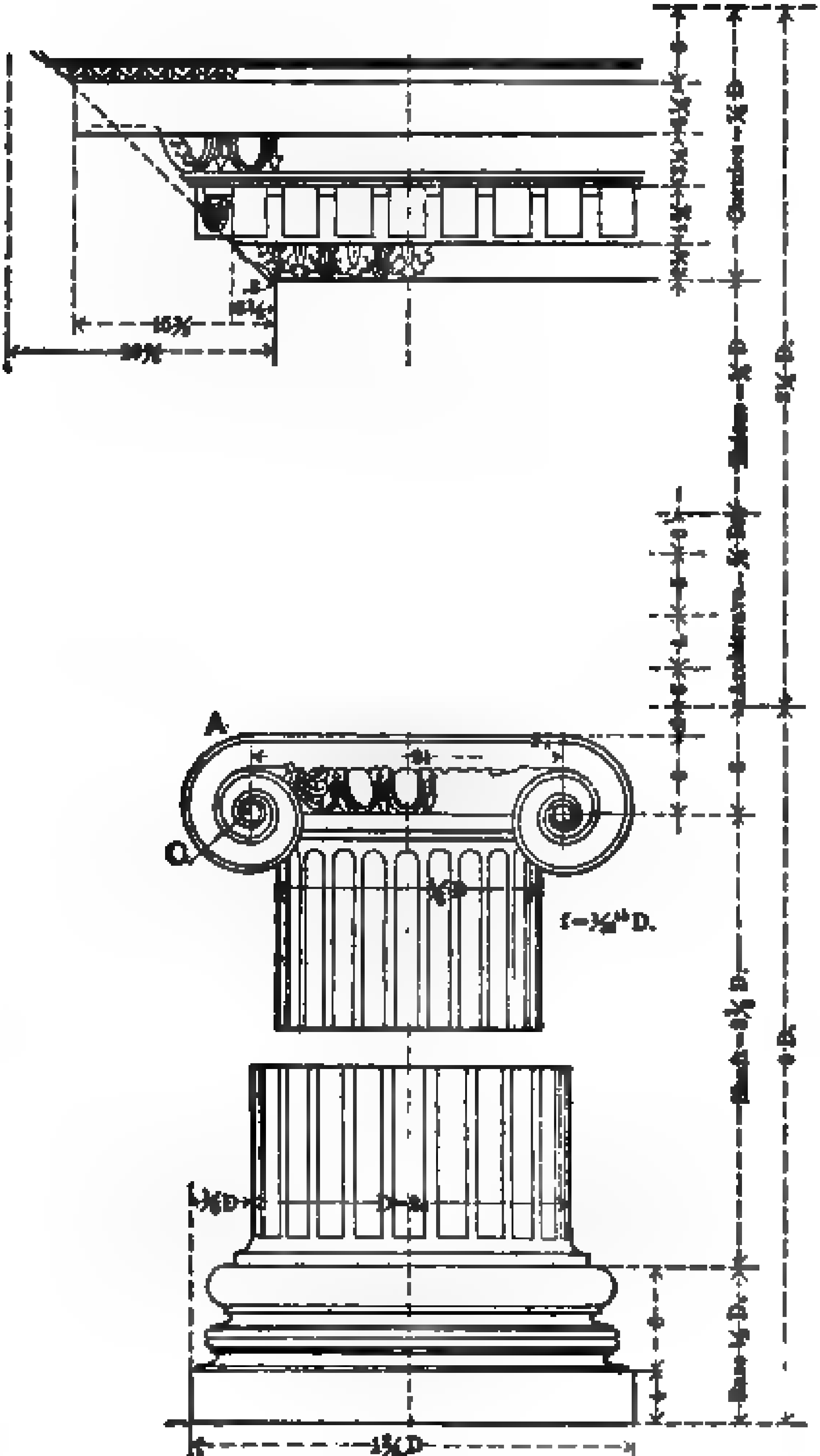
The Ionic Order. "The prototypes of the IONIC ORDER (Fig. 3) are to be ound in Persia, Assyria, and Asia Minor. It is characterized by BANDS in the rchitrave and DENTILS in the bed-mold, both of which are held to represent mall sticks laid together to form a beam or a roof. But the most conspicuous

and distinctive feature is the SCROLLS which decorate the CAPITAL of the colu
These have no structural significance, and are purely decorative forms deri
from Assyria and Egypt. Originally the Ionic order had no FRIEZE and n
ECHINUS in the capital. These were borrowed from the Doric order, and in

Dimensions are in 60ths of Diameter

Fig. 2. The Doric Order

like manner, the dentils and bands in the Doric were borrowed from the Ionic.
The Ionic frieze was introduced in order to afford a place for sculpture, and was
called by the Greeks the ZOOPHOROUS, or figure-bearer. The typical IONIC BASE
is considered to consist mainly of a SCOTIA, as in some Greek examples. It is
common, however, to use instead what is called the ATTIC BASE, consisting of



Dimensions are in 60ths of Diameter.

Fig. 3. The Ionic Order

SCOTIA and two FILLETS between two large TORUSES, mounted on a PLINTH, the whole half a diameter high. The plinth occupies the lower third, or one-sixth of a diameter. Vignola adopted for his Ionic order a modification of the Attic base, substituting for the single large scotia two small ones, separated by one or two beads and fillets, and omitting the lower torus." This is the base shown in Fig. 3. "The Ionic frieze is plain, except for the sculpture upon it. It sometimes has a curved outline, as if ready to be carved, and is then said to be PULVINATED, from pulvinar, a bolster, which it much resembles. The shaft of the column is ornamented with twenty-four FLUTINGS, semicircular in section, which are separated not by an ARRIS, but by a FILLET of about one-fourth their width. This makes the flutings only about two-thirds as wide as the Doric channels, or about one-ninth of a diameter, instead of one-sixth."

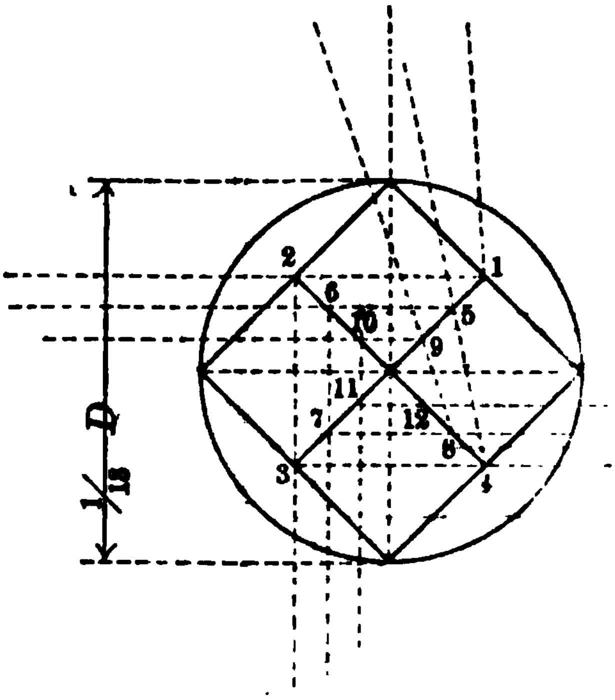
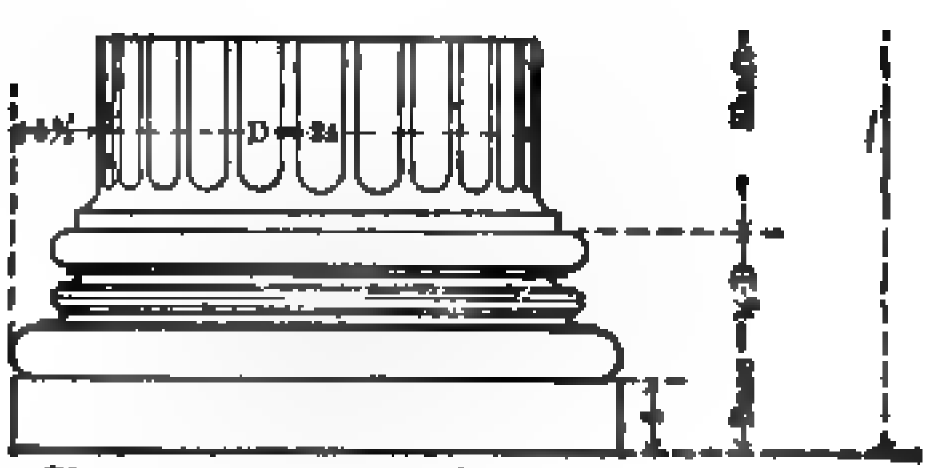


Fig. 4. The Ionic Volute

To Describe the Ionic Volute. There are several methods of doing this, the simplest being by means of centers found as shown by the diagram in Fig. 4. First locate the center of the EYE ½ *D* vertically below the point *A*, Fig. 3. Then describe a circle with a diameter equal to 1/18 *D*, to form the eye. Inside of this circle inscribe a square at 45 degrees to a horizontal; then draw the axes 1–3 and 2–4, and divide each of these into six equal parts. Then with the point 1 as a center, and a radius extending to *A*, Fig. 3, draw a quarter-circle to line 1–2 produced, with 2 as a center, continue the curve until it intersects 2–3 produced, and so on. The centers for the outer curve of the volute are at the points 1, 2, 3, 4, 5, 6, etc. For the centers for the inner curve, start with a point one-third the way from 1 to 5, then a point one-third the way from 2 to 6, and so on.

The Corinthian Order. "The three distinguishing characteristics of the CORINTHIAN ORDER (Fig. 5) are a tall, bell-shaped capital, a series of small brackets called MODILLIONS, which support the cornice instead of MUTULES, in addition to the DENTILS, and a general richness of detail which is enhanced by the use of the ACANTHUS LEAF in both capitals and modillions. Here, again, the ATTIC BASE is commonly used, but sometimes, especially in large columns, a base is used which resembles Vignola's IONIC BASE, except that it has two BEADS between the SCOTIAS instead of one, and also a lower TORUS. The SHAFT is fluted like the Ionic shaft, with twenty-four semicircular FLUTINGS, but they are sometimes filled with a convex molding or CABLE to a third of their height. Almost all the buildings erected by the Romans employ the Corinthian order."

The Composite Order. "The COMPOSITE ORDER is a heavier Corinthian, just as the Tuscan is a simplified Doric. The chief proportions are the same as in the Corinthian order, but the details are fewer and larger. It owes its name to the CAPITAL, in which the two lower rows of leaves and the CAULICOLI are the same as in the Corinthian. But the caulicoli carry only a stunted LEAF-BELL, and the upper row of leaves and the sixteen VOLUTES are replaced by the large ECHINUS, SCROLLS and ASTRAGAL of a complete Ionic capital. Vignola's composite entablature differs from his Ionic chiefly in the shape and size of the



Dimensions are in 24ths of Diameter

THE CORINTHIAN ORDER

Fig. 5. The Corinthian Order

DENTILS. They are larger, and are more nearly square in elevation, bein fifth of a diameter high and one-sixth wide, the INTERDENTIL being one-t

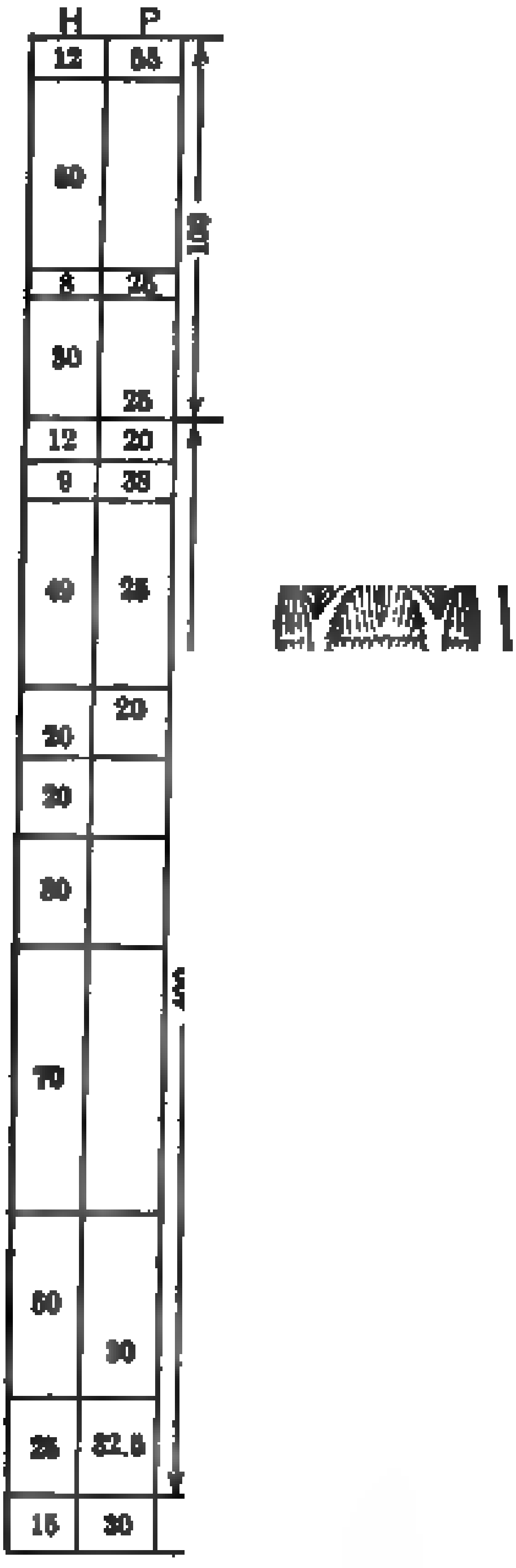


Fig. 6. An Egyptian Order. Diameter Divided into Sixty Parts

and they are set one-ha a diameter apart, on ce The composite capital is ployed in the Arch of Tit Rome, and elsewhere, wi Corinthian entablature, and BLOCK CORNICE occurs in th called FRONTISPIECE of N as well as in the temple Athens, in connection wi Corinthian capital."

Egyptian Style.* The a tecture of the ancient Egypt is characterized by boldnes outline, solidity, and grand The principal features of EGYPTIAN STYLE of architect are: uniformity of plan, n deviating from right lines angles; thick walls, having outer surface slightly deviat inwardly from the perpendic the whole building low; roofs composed of stones reaching one piece from pier to p these being supported by e mous columns, very stout proportion to their height shaft sometimes polygonal, h ing no base, but with a g variety of handsome capit the foliage of these being of palm, lotus and other lea ENTABLATURES having sim an ARCHITRAVE, crowned wit huge CAVETTO ornamented w sculpture; and the INTERCOLU NIATION very narrow, usu $1\frac{1}{2}$ diameters and seldom ceeding $2\frac{1}{2}$. A great dissi larity exists in the proporti forms and general features of Egyptian columns. For practical use the colu shown in Fig. 6 may be taken as a standard of the Egyptian style.

LIGHTNING-CONDUCTORS

Rules for the Erection of Lightning-Conductors. The following r for the erection of lightning-conductors were issued in 1882 by the Departme Explosives of the English Home Office to the occupiers of all factories and ma

* From The American House Carpenter, by R. G. Hatfield.

for explosives, and to those local and police authorities upon whom de-
es the inspection of stores of explosives:

) **Material of Rod.** Copper, weighing not less than 6 oz per ft run, the rical conductivity of which is not less than 90% of that of pure copper, er in the form of rod, tape, or rope of stout wires, no individual wire being than No. 12, Birmingham Wire-Gauge (0.109 in) the English standard -gauge. Iron may be used, but should not weigh less than 2½ lb per foot n.

) **Joints.** Every joint, besides being well cleaned and screwed, scarfed, iveted, should be thoroughly soldered.

) **Form of Points.** The point of the upper terminal * of the conductor ld not have an angle sharper than 90°. A foot below the extreme point a er ring should be screwed and soldered on to the upper terminal, in which should be fixed three or four sharp copper points, each about 6 in long. desirable that these points should be so platinized, gilded, or nickel-plated resist oxidation.

) **Number and Height of Upper Terminals.** The number of conductors or er terminals required will depend upon the size of the building, the material hich it is constructed, and the comparative height above ground of the ral parts. No general rule can be given for this, except that it may be med that the space protected by the conductor is, as a rule, a cone, the is of whose base is equal to the height of the conductor from the ground.

) **Curvature.** The rod should not be bent abruptly around sharp corners. o case should the length of a curve be more than half as long again as its d. A hole should be drilled in string-courses or other projecting masonry, n possible, to allow the rod to pass freely through it.

) **Insulators.** The conductor should not be kept from the building by or other insulators, but attached to it by fastenings of the same metal as of the conductor itself.

) **Fixing.** Conductors should preferentially be taken down the side of building which is most exposed to rain. They should be held firmly, but holdfasts should not be driven in so tightly as to pinch the conductor or ent contraction and expansion due to change of temperature.

) **Other Metalwork.** All metallic spouts, gutters, iron doors, and other es of metal about the building should be electrically connected with the uctor.

Earth-Connection. It is most desirable that, whenever possible, the r extremity of the conductor should be buried in permanently damp soil. ce, proximity to rain-water pipes and to drains or other water is desirable. a very good plan to bifurcate the conductor close below the surface of the nd, and to adopt two of the following methods for securing the escape of ightning into the earth: (*a*) A strip of copper tape may be led from the om of the rod to a gas or water-main (not merely to a leaden pipe), if such near enough, and be soldered to it; (*b*) a tape may be soldered to a sheet pper, 3 by 3 ft by 1/16 in thick, buried in permanently wet earth and sur- ded by cinders or coke; (*c*) many yards of copper tape may be laid in a h filled with coke, having not less than 18 sq ft of copper exposed.

) **Protection from Theft, etc.** In places where there is any likelihood of copper being stolen or injured, it should be protected by being enclosed

* The upper terminal is that portion of the conductor which is between the top of the e and the point of the conductor.

in an iron gas-pipe, reaching 10 ft (if there is room) above ground and a distance into the ground.

(11) **Painting.** Iron conductors, galvanized or not, should be painted. I optional with copper ones.

(12) **Inspection.** When the conductor is finally fixed it should in all c be examined and tested by a qualified person, and this should be done i case of new buildings after all work on them is finished. Periodical exami tion and testing, should opportunities offer, are also very desirable, espe when iron earth-connections are employed.

Lightning-Protection for High Chimneys. The following is a desc tion of the system of lightning-protection for the radial-brick chimney-st 350 ft in height, for the plant of the St. Joseph Lead Company, Herculan Mo.

Conductor. The conductor used is of commercially pure copper, No. Brown & Sharpe gauge, in the form of a cable, consisting of twenty-e wires, seven strands, four wires to the strand, and 5/8 in in diameter, 230 circular mils. The vertical conductors are of continuous lengths from the of the chimney to and into the ground. A circuit-conductor is placed below the top of the chimney and connected to each down-conductor by a t two-way splice.

Points. The air-terminals are eight in number equally spaced around top of the chimney, and consist of solid, copper bars 1 in in diam and 10 f length, the upper 12 in tapering to a point and covered with a 12-in thimbl genuine platinum. Air-terminals extend 5 ft above the top of the stack; the lower end of each copper bar is set in a heavy copper T coupler, which c nects the same into the circuit-conductor. Each rod is held in place by he anchor-fasteners, bolted from the inside of the stack. These anchors are cased in copper tubes set in the solid masonry.

Grounding. At a point below the ground-level and at the chimney-l the conductor is carried in a downward course from the chimney, in a tre bedded in charcoal, to a point 5 ft outside the foundation-line. An additi conductor is spliced into the main cable at this point, forming a Y w branches terminating 15 ft apart. Two well-holes are bored to a depth of proximately 20 ft into permanent moisture. The end of each Y conducto electrically soldered into perforated, copper reservoirs 4½ in in diam and 1 in length, and filled with pea-size charcoal. The effect of the reservoir is give the required amount of surface-contact with the earth and to insure p manent moisture through the charcoal by capillary attraction. Each m conductor is thus grounded in two places instead of in one place.

Lead Covering. To preserve the conductor system against decomposi in ozone, in which sulphuric or other acid gases may exist, all of the condu system at the top, and to a point 75 ft below the top of the chimney is cove with lead 1/8 in in thickness. Exception is made to the platinum-covered t top of each rod, which requires no lead covering. Where splices are made anchor-fasteners set, the whole is covered with lead sleeves or hoods thorou wiped and hermetically sealed. Connections of point-bar T's etc., are soldered, lead-covered and sealed. Practical experience seems to show t all lightning-conductor systems on chimneys should be lead-covered hermetically sealed to a point, approximately 25 ft downward from the top protect the copper against decomposition, not necessarily as thick as on chimney, but, say, 1/16 in, the thickness being determined by the size and s

of the stack. It has been found that in from three to five years there is a decided honeycombing of the copper, through the action of the sulphuric and other acid gases. It has often been necessary to replace points, sections of cable, etc., entirely eaten away from this cause.

INTERPHONES. AUTOMATIC TELEPHONES FOR INTERCOMMUNICATING SERVICE

Description. The interphone system is an application of the telephone for interior use. It is an automatic, intercommunicating system, requiring neither switchboard nor operator, and being self-contained within the walls of the establishment for whose benefit it has been installed.

Advantages. In brief, the advantages of such a system are these: (1) the mere pressing of a button gives a person telephone-connection with any desired party, without the loss of time involved in first calling up a third party; (2) recourse to directory or information bureau is made unnecessary through the use of labels, properly inscribed, on the face of the instrument; (3) no maintenance-expense is involved, and the system, consequently, is as inexpensive to operate as an electric door-bell; (4) the wiring-arrangement is such that the system may be provided for when the original plans for a new building are being drawn up, and in this respect it does not differ much from a system of electric lights or plumbing.

The Use of Interphones in residences, schools, hospitals, factories, mills, offices, stores and clubs is constantly increasing. The same general features apply to all of these types of installations, and in practically every case it is the simplicity of the system that especially recommends it for service. The interphone usually fits in where formerly call-bells, speaking-tubes, messenger service and other inadequate methods were the rule. The interphone field of service is in the establishment whose needs call for from four to thirty-two telephone-stations. When there are more than thirty-two the installation of a private telephone-exchange, with a switchboard, is better practice.

Types of Interphones. There are several types of interphones for varying degrees of service.

(1) The most familiar instrument is a wall-interphone, of the NON-FLUSH TYPE. The telephone is of metal, with connecting buttons, labels, bells, mouthpiece, hook and receiver, all mounted on its face. This instrument is to be attached directly to the wall.

(2) The FLUSH TYPE resembles the first-mentioned type in every particular but the one implied in its title. The instrument is mounted into the wall, with its face flush with the rest of the wall-surface. These two instruments are most popular for installation in club-hallways, in stores and factories, in residences, and in all places where wall-telephones would ordinarily be used. Busy offices and stores often employ variations of types (1) and (2) and use a DESK-SET, a separate instrument taking care of the connecting buttons and labels, or a HAND-SET.

(3) The DESK-STAND telephone is of the type often used for local and long-distance service. Connected with it is a metal box containing the rows of buttons and labels, each label being opposite the button through which is secured connection with the corresponding station. The telephone in this case stands on the desk, and the key-box is conveniently close at hand, either on the desk or on the wall.

(4) Some prefer for this service the HAND-SET, with the receiver and transmitter in one piece. This is a convenient, compact instrument, well fitted for use in an office.

(5) From two to six instruments of still another type make up a PARTY-LINE INTERPHONE SYSTEM. Here there are no connecting buttons, the principle involved being the same as that of the elementary, farmers' line. This makes a convenient private-line system for a small residence, and is appropriate for a house-to-garage circuit.

Variations from Standard Types. There are systems with variations from the standard types. Many schools are using a combination of interphones of type (1) or (2) with (5). In the principal's office is an instrument of type (1) or (2) with a connecting button for each outside station, while the classroom-telephones are all of type (5). With this system the principal can at any time call up any teacher; but a teacher can call up another classroom only through the medium of this MASTER-STATION, which acts as a sort of exchange. The advantages of this arrangement for a school are obvious. In a hospital the instruments are usually placed outside of the more important operating-rooms and wards and in the offices and reception-rooms.

Wiring and Batteries. All wiring is enclosed in cables. Energy is obtained from dry cells. The only maintenance-expense connected with an interphone system is the occasional renewal of these batteries.

VACUUM-CLEANING

General Description. Vacuum-cleaners are appliances which have come into use during recent years and which are for the purpose of removing dirt and dust from rooms of buildings, cars, steamships, etc., or from furniture, carpets, curtains, or other interior fittings. The dust and dirt are removed by suction and the apparatus consists of an air-pump which is arranged to draw the air and the dirt or dust contained in it through pipe and nozzle. The nozzle is drawn or passed over the surfaces which are to be cleaned. Screens of muslin or other appropriate cloth are used to separate by filtration the dust and dirt which are borne along with the stream of air; and in some types of apparatus this process is assisted by what are called baffle-plates which are added to make the heavier particles of dust drop by their own weight to the lower part of the receptacle placed to receive them. About the year 1890 compressed air was used for the first time in railroad-cars for purposes of cleaning and dust-removal. There were serious objections to this method of cleaning, however, as it was found that the jets of compressed air blew out the dust and dirt in such a way that it was difficult to arrange for their collection and retention; the principle of suction was consequently introduced to overcome these difficulties.

Types of Vacuum-Cleaners. The machines belonging to the earliest types usually consist of a pump, the motor-power of which is either a gas-engine or an electric motor, the machines being portable. They can be moved about from one building to another as occasion demands. Cleaners of the next type introduced involve an installation in the basement or lower part of a building and a fixed and permanent position. From the central plant pipes are run to various rooms and apartments and are fitted in such rooms or apartments or in adjacent halls or corridors, with valves to which are attached the hose with the cleaning appliances at the end. In some cases this vacuum-arrangement is combined with another for washing floors, the secondary system including a second set of pipes from a tank filled with soap and water. Compressed air is employed to spray the latter over the floor, and both dirt and water are finally removed

through pipes to the street-sewers. A portable tank is used for the soap and water. Vacuum-cleaners of a third type consist of small machines which take the place of the brooms and dusters or are used in connection with them. They are now very generally used and may be driven by an electric motor, by foot, or by hand. These last-mentioned, smaller, portable cleaners are used for many other purposes than the ordinary cleaning of rooms and furniture.

Details and Specifications for Vacuum-Cleaning Installations. Complete plans and specifications for the installation of a vacuum-cleaning plant for a building may be obtained from any of the numerous manufacturers making such apparatus and taking contracts to put it in place. There are several types of machines and systems of installation and detailed descriptions would exceed the limits of space in this handbook.

WATERPROOFING FOR FOUNDATIONS *

The Waterproofing of Substructure Work is, comparatively speaking, a modern branch of engineering. It is only within recent years that it has become necessary to construct deep basements for buildings. In the past, the more important structures, such as cathedrals, capitols, state-buildings and the like, were usually built upon high ground, and water was prevented from entering the basements of such buildings by means of drainage. Waterproofing, as we now know it, was generally unnecessary. With the advent of the so-called sky-scrapers, however, requiring large mechanical plants, deep basements became an actual necessity, and as these basements are usually carried below ground-water level, and in many instances below tide-level, the question became one of utmost importance. Like almost every detail of a modern building, waterproofing is a specialty. Each building presents its own problems, and the safest plan is to leave the solution of these problems to some one expert in the knowledge of waterproofing who has made it a special study and knows how best to overcome the existing difficulties. It may be laid down as an invariable rule that, where conditions are at all serious, the owner or the general contractor will save money in the long run if he employs the services of an expert waterproofer to place his waterproofing-seal, regardless of the method he wishes to use.

Pressure-Resistance Versus Waterproofing. In waterproofing large basements where actual pressure exists, it is a question for the engineer to decide whether it is more economical to attempt to secure an absolute PRESSURE-JOB or a WATER-PROOF JOB in connection with a drainage system. As a general rule, it may be stated that where a building is generating its own power, it is more economical to use a drainage system with an open sump than to construct a pressure-cellar, the cost of pumping being much less than the interest charges on the cost of a floor-slab sufficiently strong to withstand the pressure.

Waterproofing Concrete Foundations. The three following subdivisions of this subject, discussing the causes of permeability of concrete, the addition of substances to render it more water-proof, and the treatment of its surfaces to make it less permeable, embody the conclusions of Committee D–8 of the American Society for Testing Materials.† This committee, since its organization in 1905 has, through laboratory-tests and experiments, together with examinations of work during construction and after completion, as well as the study of literature on the subject, sought to secure sufficient information to enable it to for-

* For foundations in general, see Chapter II.

† This article, to the middle of page 1713, is the substance of a Report submitted to the American Society for Testing Materials at its meeting, June 24–28, 1913. This Society has (1920) no Standard Specifications for Waterproofing, but published in 1918 our Tentative Specifications on this subject.

mulate definite methods for securing water-proof concrete structures. The work of the committee was complicated by reason of the facts that there seemed to be so little concordance between results of tests obtained under laboratory-conditions and in the field and that it was necessary to extend its investigations over a period of years in order to determine the permanency of the action noted. The committee reported that while it had not been able to arrive at sufficiently definite conclusions to enable it to formulate specifications for the making of concrete structures water-proof or for materials to be used in such work, it had reached certain general conclusions which might be of assistance to the constructor in securing the desired result of impermeable concrete. Early in the investigation, the work was found to subdivide naturally into three branches, and the conclusions reached will be grouped in order under these subdivisions, which are:

(1) The determination of causes of the permeability of concrete as usually made from mixtures of Portland cement, sand and stone, or other coarse aggregate, in proportions of from 1 of cement, 2 of sand and 4 of stone, to 1 of cement, 3 of sand and 6 of stone, and the best methods of avoiding these causes.

(2) The rendering of concrete more water-proof by adding to ordinary mixtures of cement, sand and stone, other substances which, either by their void-filling or repellent action, would tend to make the concrete less permeable.

(3) The treatment of exposed surfaces after the concrete or mortar has been put in place and hardened more or less, either by penetrative, void-filling or repellent liquids, making the concrete itself less permeable; or by extraneous protective coatings, preventing water from having access to the concrete.

Considering these several subdivisions separately and in the order named, the committee arrives at the following conclusions:

(1) **Causes of Permeability of Concrete.** In the laboratory and under test-conditions where properly graded and sized coarse and fine aggregates are used, in mixtures ranging from 1 of cement, 2 of sand and 4 of stone, to 1 of cement, 3 of sand and 6 of stone, impermeable concrete can invariably be produced. Even with sand of poor granulometric composition, with mixtures as rich as 1 of cement, 2 of sand and 4 of stone, permeable concrete is seldom, if ever, found and is a rare occurrence with mixtures of 1 of cement, 3 of sand and 6 of stone. But the fact remains, nevertheless, that the reverse often obtains in actual construction, permeable concretes being encountered even with mixtures of 1 of cement, 2 of sand and 4 of stone and are of frequent occurrence where the quantity of the aggregate is increased. This the committee attributes to:

(a) Defective workmanship, resulting from improper proportioning, lack of thorough mixing, separation of the coarse aggregate from the fine aggregate and cement in transporting and placing the mixed concrete, lack of density through insufficient tamping or spading, improper bonding of work-joints, etc.

(b) The use of imperfectly sized and graded aggregates.

(c) The use of excessive water, causing shrinkage-cracks and formation of laitance-seams.

(d) The lack of proper provision to take care of expansion and contraction, causing subsequent cracking.

Theoretically, none of these conditions should prevail in properly designed and supervised work, and they are avoided in the laboratory and in the field, under test-conditions, where speed of construction and cost are negligible items, instead of being governing features as they must be in actual construction. Properly graded sands and coarse aggregates are rarely, if ever, found in nature in sufficient quantities to be available for large construction, and the effect of poorly graded

aggregates in producing permeable concrete is aggravated by poor and inefficient field-work. Even if the added expense of screening and remixing the aggregates could be afforded, so as to secure proper granulometric composition to give the density required to make untreated concretes impermeable, it is seemingly often a commercial impossibility on large construction to obtain workmanship even approximating that found in laboratory-work.

(2) **Addition of Foreign Substances to Cement Before or During Mixture.** The committee finds that in consequence of the conditions outlined above, substances calculated to make the concrete more impermeable, either incorporated in the cement or added to the concrete during mixing, are often used. This has resulted in the development and placing on the market of numerous patented or proprietary waterproofing-compounds, the composition of which is more or less of a trade-secret. While it has been impossible for the committee to test all of the special waterproofing-compounds being placed on the market, it has investigated a sufficient number of these, as well as the use of certain very finely divided, naturally occurring or readily obtainable commercial mineral products, such as finely ground sand, colloidal clays, hydrated lime, etc., to form a general idea of the value of the different types. The committee finds:

(a) That the majority of patented and proprietary integral compounds tested have little or no immediate or permanent effect on the permeability of concrete and that some of these even have an injurious effect on the strength of mortar and concrete in which they are incorporated.

(b) That the permanent effect of such integral waterproofing-additions, if dependent on the action of organic compounds, is very doubtful.

(c) That in view of their possible effect, not only upon the early strength, but also upon the durability of concrete after considerable periods, no integral waterproofing-material should be used unless it has been subjected to long-time practical tests under proper observation to demonstrate its value, and unless its ingredients and the proportion in which they are present are known.

(d) That in general, more desirable results are obtainable from inert compounds acting mechanically, than from active chemical compounds whose efficiency depends on change of form through chemical action after addition to the concrete.

(e) That void-filling substances are more to be relied upon than those whose value depends on repellent action.

(f) That, assuming average quality in sizing of the aggregates and reasonably good workmanship in the mixing and placing of the concretes, the addition of from 10 to 20% of very finely divided void-filling mineral substances may be expected to result in the production of concrete which, under ordinary conditions of exposure, will be found impermeable, provided the work-joints are properly bonded, and cracks do not develop on drying, or through change in volume due to atmospheric changes, or by settlement.

(3) **External Treatment.** While external treatment of concrete would not be necessary if the concrete itself, either naturally or by the addition of waterproofing-material, was impermeable to water, it has been found in practice that in large construction, no matter how carefully the concrete itself has been made, cracks are apt to develop, due to shrinkage in drying out, expansion and contraction under change of temperature and moisture-content, and through settlement. It is, therefore, often advisable in important construction to anticipate and provide for the possible occurrence of such cracks by external treatment with a protective coating. Such coating must be sufficiently elastic and cohesive to prevent the cracks extending through the coating itself. The application of merely penetrative void-filling liquid washes will not prevent the passage of

water due to cracking of the concrete. The committee has, therefore, considered surface-treatment under two heads:

(a) Penetrative void-filling liquid washes.

(b) Protective coatings, including all surface-applications intended to prevent water coming in contact with the concrete.

Penetrative Washes. While some penetrative washes may be efficient in rendering concrete water-proof for limited periods, their efficiency may decrease with time and it may be necessary to repeat such treatment. Some of these washes may be objectionable, due to discoloring the surface to which they are applied. The committee, therefore, believes that the first effort should be made to secure a concrete that is impermeable in itself and that penetrative void-filling washes should only be resorted to as a corrective measure.

Protective Coatings. While protective extraneous bituminous or asphaltic coatings are unnecessary, so far as the major portion of the surface of the concrete is concerned, provided the concrete, either in itself or through the addition of integral compounds, is made impermeable, they are valuable as a protection where cracks develop in a structure. It is therefore recommended that a combination of inert void-filling substances and extraneous waterproofing be adopted in especially difficult or important work.

Bituminous or Asphaltic Coatings. Considering the use of bituminous or asphaltic coatings, the committee finds:

(a) That such protective coatings are often subject to more or less deterioration with time, and may be attacked by injurious vapors or deleterious substances in solution in the water coming in contact with them.

(b) That the most effective method for applying such protection is either the setting of a course of impervious brick dipped in bituminous material into a solid bed of bituminous material, or the application of a sufficient number of layers of satisfactory membranous material cemented together with hot bitumen.

(c) That their durability and efficiency are very largely dependent on the care with which they are applied. Such care refers particularly to proper cleaning and preparation of the concrete to insure as dry a surface as possible before application of the protective covering, the lapping of all joints of the membranous layers, and their thorough coating with the protective material. The use of this method of protection is further desirable because proper bituminous coverings offer resistance to stray electrical currents, the possible attack from which is referred to in succeeding paragraphs.

Rich Mixtures. So far, the committee has considered only concretes of the usual proportions, namely, those ranging from 1 of cement, 2 of sand and 4 of stone, to 1 of cement, 3 of sand and 6 of stone. It has been suggested that impermeable concretes could be assured by using mixtures considerably richer in cement. While such practice would probably result in an immediate impermeable concrete, it is believed by many that the advantage is only temporary, as richer concretes are more subject to check-cracking and are less constant in volume under changes of conditions of temperature, moisture, etc. Therefore, the use of more cement in mass-concrete would cause increased cracking, unless some means of controlling the expansion and contraction is discovered. With reinforced concretes the objection is not so great, as the tendency to cracking is more or less counteracted by the reinforcement.

Fine Flour Mixtures. It has also been suggested that the presence in the cement of a larger percentage of very fine flour might result in the production of a denser and more impermeable concrete, through the formation of a larger amount of colloidal gels. Neither of these suggestions has been especially investigated by the committee. Both appeal to the committee, however, for the

reason that they substitute active cementitious substances for the largely inactive void-filling materials previously recommended, thus increasing the strength of the concrete.

Character of Workmanship. In conclusion, the committee would point out that no addition of waterproofing-compounds or substances can be relied upon to completely counteract the effect of bad workmanship, and that the production of impermeable concrete can only be hoped for where there is determined insistence on good workmanship.

Saline Waters. Electrical Action. The production of impermeable concrete has assumed greater importance since the appointment of this committee, owing to the well-known injurious action of saline or alkaline waters and to the suggested possible effect of the moisture in concrete occasioning or aggravating electrical action from stray currents. Originally, the question of waterproofing involved mainly the physical troubles resulting from water passing through concrete without any special consideration of its effect on its durability, other than a gradual leaching out of the cement. Recent developments suggest the possibility that, owing to the increased conductivity of damp concrete to electrical currents, such currents, if present, may so affect damp concrete as to seriously lessen its integrity; and this possibility further emphasizes the importance of the recommendation that no waterproofing-compound of unknown chemical composition be added to concrete, as recent tests seem to show that the action of electrical currents is aggravated by the presence of certain solutions.

Waterproofing by External Linings of Brick, Tar, or Asphalt, and Felt. The oldest method of waterproofing is the one involving the use of a tar-and-felt or asphalt-and-felt seal (Fig. 1). This consists of building first a supporting wall and a supporting concrete slab to hold the seal. On the floors, this slab is usually composed of concrete, 4 in thick. The walls are generally of brick from 4 to 8 in thick, but occasionally 4-in terra-cotta tiles are used. Upon this base a swabbing of tar or asphalt is placed and before this has become cold or set, one thickness of paper, saturated with coal-tar, is laid. This paper receives a swabbing of coal-tar and asphalt and another layer of paper is placed, the operation being continued until there are three or more layers of paper with four or more swabbings of the tar or asphalt. For damp-proof work, three layers of paper with four swabbings of tar are usually sufficient. For waterproofing-work not less than five and usually six layers of paper with from six to seven swabbings of tar are used. The main walls of the structure are then built against the wall-waterproofing, and after these are in place, the main concrete basement-floor is laid immediately on top of the floor-seal, the idea being to form a continuous water-proof seal enveloping the entire basement below grade. The difficulties of this system consist chiefly in securing perfect laps at all points in the work, and unless extreme care is used and unless there is perfect coöperation between the waterproofer and the mason-contractor, there is apt to be a break somewhere in the seal, usually where the wall-waterproofing is supposed to be joined to the floor-work. The disadvantages of this system are due to the fact that the seal is not permanent in all soils as the subsurface water frequently contains acids which destroy the seal. Then again, the seal may be easily punctured by the mason-contractor in building his wall against it or in laying the concrete floor upon the flat work. The chief disadvantage, however, is that the waterproofing-seal is invariably buried behind a mass of masonry, either brick or concrete, which means that should there be a leak, due to either carelessness or accident, through the waterproofing-seal, it is frequently impossible to stop it. It not infrequently happens that when a leak has developed in tar-and-felt work, the actual presence of the water does not show opposite the leak,

but following some line of least resistance, appears from 50 to 100 ft, or more away from where the actual damage causing the leak occurs. In actual waterproofing work it is seldom attempted to secure a bottle-tight job with tar and felt. Instead, some system of drainage is installed beneath the water-proof seal which is on the floors of the building, and the water is conducted through the

Fig. 1.* Felt-Waterproofing for Foundations

or other pipes to some central sump from which it is mechanically pumped to a sewer. The purpose of the waterproofing in this case, therefore, is to concentrate or drive the water to this sump. For shallow cellars and especially dampproofing-work, this tar-and-felt method is the most economical and most frequently employed.

Waterproofing by Coating with Water-Proof Cement. For deep and difficult work a comparatively new method of waterproofing is often used (Fig 2). This consists of placing a coating of water-proof cement upon the interior surface of the exterior walls of the building and over the upper surface of the concrete floor-slab in the basement or subbasement. Fig. 3 shows a foundation for an engine, the concrete being waterproofed as shown. The pit is made somewhat larger than the foundation, the extra space being filled in with cinders, dry bricks or terra-cotta blocks, which may be readily removed to allow access to the bed-plate bolts for which hand-holes have been cast in the concrete, thus permitting the complete removal of the engine. The figure

* Reproduced, by permission, from a pamphlet published by The Waterproofing Company New York, and showing the greater thickness of walls and floor required for the outside-surface brick-and-felt method of waterproofing as compared with the inside-surface waterproof-cement coating. Taken from design for waterproofing in a prominent New York building. See, also, Fig. 2.

s a 2-in sand cushion and a 2-in layer of planks under the engine-foundation. is not a part of the waterproofing but is put in to prevent the communica- of vibration. Fig 4 shows reinforced-concrete floors for an engine-room boiler-room, the concrete slab being 12 in thick under the former and 24 in

Fig. 2.* Cement Waterproofing for Foundations

under the latter. Both floors are covered with a 1-in course of water-proof nt. The reinforcement is put in as shown and in sizes and spacing as rs:

12-in slab	24-in slab
Rods in two courses er rods, 4 in on centers, 6 in from urface er rods, 6 in on centers, 2 in from rface five rods, total area of cross-section 0.703 sq in; per square foot of sur- ce, 2.39 lb	Rods in three courses Lowest rods, 3 in on centers, 12 in from surface Intermediate rods, 3 in on centers, 7 in from surface Upper rods, 6 in on centers, 2 in from surface For ten rods, total area of cross-section, 1.4 sq in; per square foot of surface, 4.78 lb

rom a pamphlet published by The Waterproofing Company, New York, and show- luced total thickness of walls and floor required for the inside-surface water-proof t method of waterproofing. Taken from design for waterproofing of the same ng shown in Fig. 1. The walls and floors were put in place in the monolithic form.

There are many compounds advertised to make cement or concrete wat-
Besides these, there are water-proof cements manufactured by secret pr-
and applied by companies that make a specialty of waterproofing. S-
the many waterproofing-compounds have merit; but the main factor-

Fig. 3.* Engine-foundation with Water-proof Cement

successful job of waterproofing are the skill and experience of the waterpr-
who do the work. It is claimed that to apply cement waterproofing so a-
obtain efficient results requires more skill than to apply a tar-and-felt sea-
a cement waterproofing, once properly applied, seems to possess some adva-

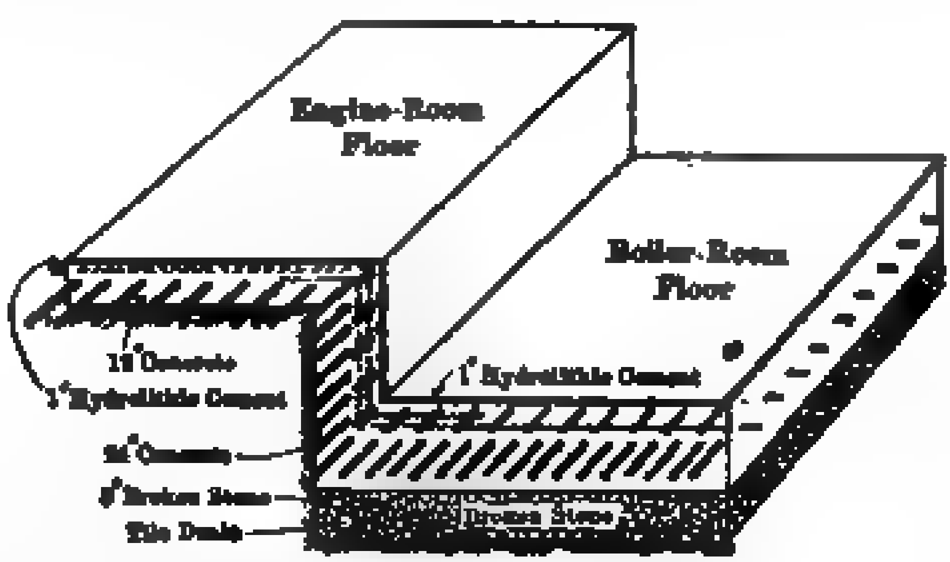


Fig. 4.* Reinforced-concrete Floor with Water-proof Cement

over the older method of tar and felt. One advantage is that the waterpr-
is accessible, and that if any leaks develop, they are apparent and can be -
and economically repaired by cutting out the old waterproofing and p-
a new coating where the damage exists. Another advantage claimed -
cement waterproofing is generally permanent and not damaged by the -

* Reproduced by permission of The Waterproofing Company, New York.

found in solution with water in soil. By the cement method the cost of rick supporting walls and the concrete supporting slab is eliminated as is he corresponding cost of the necessary excavation for them; and finally, aterproofing on the floor serves the double purpose of waterproofing and ng-surface, thus saving the cost of the cement finish usually found in nents and subbasements. One of the disadvantages of cement water- ing is that the material is rigid and is fractured by any settlement of the ing or contraction in the concrete upon which it is placed. Experience has n, however, that settlement-cracks usually take place before the water- ing contractor has left the building and that there is little or no trouble these causes after his work is completed. Contraction-cracks in concrete, ver, seem to develop at any time within twenty-four months after concrete een placed. In order to prevent these cracks, users of the cement water- ing have adopted a system of reinforcement in the concrete, and it is claimed this reinforcement is, in the long run, an economy, as it permits of less con- and gives a better and stronger floor or wall. On brick and stone walls ouble is experienced from contraction and expansion. It should be re- bered that this work is all below grade where contraction and expansion educed to a minimum, regardless of the materials used.

aterproofing by Adding Substances to Cement. This is another od of waterproofing now being advocated by some. If this method could ys be made efficient, it would be highly advantageous. It is claimed by nanufacturers of these compounds that in order to secure a water-proof nent, for example, a certain percentage of the compound is to be mixed the cement before it is incorporated in the concrete. The opponents of method claim, however, that it is impossible to construct a basement in way without incurring the danger of serious leaks at the joinings of one work with that of another; that leakage at these points of cleavage may creased by the use of waterproofing-compounds; and that their principal is that they produce a very dense mass of concrete. It is always difficult nd old concrete to new, and if concrete is made water-proof, or, in other s, nonabsorbent, the difficulty of joining new concrete to a nonabsorbent of old concrete is increased. This method is effective, however, and is to commended in work which can be carried on without interruption, such, for nce, as small elevator-pits or small swimming-pools, where the concrete e started in the morning and completed by night or before any part of the has had time to attain its initial set.

FORCE OF THE WIND

lation Between the Pressure and Velocity of Wind. According to riments made in 1890 or thereabouts, by C. F. Marvin, United States l Service, the relation between wind-pressure and velocity is given very ately by the formula $p = 0.004\ V^2$, where p is the pressure in pounds per re foot on a flat surface normal to the direction of the wind, and V the ity of the wind in miles per hour. Smeaton considered the pressure as l to $0.005\ V^2$. The following table, based on Marvin's formula,* is quoted urneaure and Ketchum.†

* Marvin's formula is written $p = 0.0032\ V^2$ the values in this table will be slightly ged. See Chapter XXVII, pages 1052 and 1053; Chapter XXX, page 1199; and also 1394. [1] The formula used by the United States Signal Service is $p = 0.004\ V^2$. The pressure is probably somewhere between $0.005\ V^2$ and $0.004\ V^2$, near the former for low velocities and near the latter for high velocities.

† ee, also, Trautwine's Pocket-Book, page 321.

Table Showing the Force of the Wind

Miles per hour	Feet per minute	Feet per second	Force, in pounds, per square foot	Description
1	88	1.47	0.004	Hardly perceptible
2	176	2.93	0.014	Just perceptible
3	264	4.40	0.036	
4	352	5.87	0.064	Gentle breeze
5	440	7.33	0.1	
10	880	14.67	0.4	Pleasant breeze
15	1 320	22.0	0.9	
20	1 760	29.3	1.6	Brisk gale
25	2 200	36.6	2.5	
30	2 640	44.0	3.6	High wind
35	3 080	51.3	4.9	
40	3 520	58.6	6.4	Very high wind
45	3 960	66.0	8.1	
50	4 400	73.3	10.0	Storm
60	5 280	88.0	14.4	Great storm
70	6 160	102.7	19.6	
80	7 040	117.3	25.6	Hurricane
100	8 800	146.6	40.0	

COPIES OF TRACINGS

Blue-Prints from Tracings. The following directions * cover the wh ground. The sensitized paper can be procured, all prepared, at stores wh artists' materials are sold, so that the process of preparing the paper by me of chemicals can then be omitted. The materials required are as follows:

(1) A board a little larger than the tracing to be copied. The draw board on which the drawing and tracing are made can always be used.

(2) Two or three thicknesses of flannel or other soft white cloth, which is be smoothly tacked to the board to form a smooth surface, on which to lay t sensitized paper and tracing while printing.

(3) A plate of common double-thick window-glass, of good quality, sligh larger than the tracing to be copied. The function of the glass is to keep t tracing and sensitized paper closely and smoothly pressed together wh printing.

(4) The chemicals for sensitizing the paper. These consist simply of eq parts, by weight, of citrate of iron and ammonia, and red prussiate of pot and can be obtained at any drug-store. The price should not be over $ 10 cts per ounce for each.

(5) A stone or yellow-glass bottle to keep the solution of the above chemic in. If there is but little copying to do, an ordinary glass bottle will do, and t solution can be freshly made whenever it is wanted for immediate use.

(6) A shallow earthen dish in which to place the solution when using it. common dinner-plate is as good as anything for this purpose.

(7) A soft paste-brush, about 4 in wide.

(8) Plenty of cold water in which to wash the copies after they have b exposed to the sunlight. The outlet of an ordinary sink may be closed placing a piece of paper over it with a weight on top to keep the paper do and the sink filled with water, if the sink is large enough to lay the copy

* Taken from The Locomotive.

If it is not, it is better to make a water-tight box 5 or 6 in deep, and 6 in wider and longer than the drawing to be copied.

(9) A good quality of white book-paper.

The following directions are to be followed:

Dissolve the chemicals in cold water in these proportions: 1 oz of citrate of iron and ammonia; 1 oz of red prussiate of potash; and 8 oz of water. They may all be put into a bottle together and shaken up. Ten minutes will suffice to dissolve them.

Lay a sheet of the paper to be sensitized on a smooth table or board, pour a little of the solution into the earthen dish or plate, and apply a good even coating of it to the paper with the brush. Then tack the paper to a board by two adjacent corners, and set it in a dark place to dry. One hour is sufficient for the drying. Place the paper, with its sensitized side up, on the board on which you have smoothly tacked the white flannel cloth; lay the tracing to be copied on top of it; on top of all lay the glass plate, being careful that paper and tracing are both smooth and in perfect contact with each other, and lay the whole thing out in the sunlight. Between eleven and two o'clock in the summer-time, on a clear day, from 6 to 10 minutes will be sufficiently long to expose it; at other seasons a longer time will be required. If the location does not admit of direct sunlight, the printing may be done in the shade, or even on a cloudy day; but from 1 to 2½ hours will be required for exposure. A little experience will soon enable any one to judge of the proper time for exposure on different days. After exposure, place the print in the sink or trough of water before mentioned, and wash thoroughly, letting it soak from 3 to 5 minutes. Upon immersion in the water, the drawing, hardly visible before, will appear in clear white lines on a dark-blue ground. After washing, tack up against the wall, or other convenient place, by the corners, to dry. This finishes the operation, which is very simple and thorough. After the copy is dry, it can be written on with a common pen and a solution of common soda, which makes a white line.

Alternate Recipe for Making Blue-Prints. The following is an alternative recipe to the one given above. The paper should be prepared by floating it for one minute in a solution of ferricyanide of potassium (red prussiate of potash), 1 oz, and water, 5 oz. It should then be dried in a dark room, afterwards exposed beneath the negative until the dark shades have assumed a deep blue color, and immersed in a solution of water, 2 oz, and bichloride of mercury, 1 gr. The print should be washed, immersed in a hot solution of oxalic acid, 4 drm, and water, 4 oz, washed again and dried. Where a copy of a drawing is to be made the prepared paper is placed, sensitive side uppermost, on a flat board covered with two or three thicknesses of blanket or its equivalent. A tracing of the drawing is made, laid on the sensitized paper and held in place by a sheet of glass clamped to the board. The sensitized paper is exposed to the sunlight from 4 to 10 minutes or to a clear sky from 20 to 30 minutes and then removed, washed and dried. The only requisite as to paper is that it must stand washing. Prepared paper may be purchased.

Black-Line Copies from Tracings.* The directions for making the sensitizing solution used in this process are as follows: Dissolve separately, gum arabic, 13 dr, in 17 oz water; tartaric acid, 13 dr, in 6 oz 6 dr water; persulphate of iron, 8 dr, in 6 oz 6 dr of water. Pour the third solution into the second, stir thoroughly, and then pour the resulting mixture into the first, the stirring being continued. When the mixture is complete, add slowly, still stirring, 3 fl oz and 3 drm of liquid perchloride of iron; filter into a bottle and keep in the dark. Use a strong well-sized paper, apply a thin, smooth coat of the solution with a large brush or sponge, and then dry in a dark room with moderate heat. The

* There is as yet no known recipe resulting in jet-black lines.

paper should be yellowish in tint and will not keep long. Place the tracing made with very black ink, in the printing-frame, the drawing being in close contact with the glass, and place over it the sensitized paper, with the prepared side in contact with the back of the tracing. After an exposure of 10 or 12 minutes, the print should show a yellow drawing on a white ground. Take the print from the frame and float for a minute, face down, in a developing bath of gallic acid, or tannin, from 31 to 46 gr; oxalic acid, 1¼ gr; and water, 34 oz. Then plunge it in clear water, rinse well and dry. The orange-yellow lines will be changed into a purplish-black.

Brown-Line Copies from Tracings. The directions for making the sensitizing solution used in this process are as follows: Dissolve gelatine, 6 gr; water, 1 oz; swell in cold water, give water-bath and add tartaric acid, 8 oz; silver nitrate, 9 gr; and ammonia citrate of iron, 40 gr. Filter in a subdued light. Print in a bright light until slightly darker than ordinary printing-out paper; wash for 5 minutes; immerse in a 2½% solution of hypo until desired color is obtained; and wash and dry. Blue-prints may be turned to a rich-brown color by immersing in a solution of caustic soda the size of a bean dissolved in 5 oz of water, until the blue has changed to orange-yellow. They are then washed thoroughly, immersed in a bath of water in which has been dissolved a heaping teaspoonful of tannic acid, rinsed in clear water and dried. Paper may be sized for brown-prints by soaking it in a mixture of 90 gr of arrowroot and 5 oz of cold water, rubbed into a cream and mixed with 20 gr of glucose and 5 oz of hot water. The mixture should be boiled 2 minutes and then permitted to cool before use.

HORSE-POWER,* PULLEYS, GEARS, BELTING AND SHAFTING

Horse-Power. A horse can travel 400 yd at a walk in 4½ min, at a trot in 2 min, and at a gallop in 1 min; he occupies at a picket 3 ft by 9 ft; and his average weight is 1 000 lb. An AVERAGE HORSE carrying 225 lb can travel 25 miles in a day of 8 hr. A DRAUGHT-HORSE can draw 1 600 lb 23 miles a day, weight of carriage included. In a HORSE-MILL a horse moves at the rate of 3 ft in a second. The diameter of the track should not be less than 25 ft.

A Horse-Power, in Machinery, is estimated at 33 000 lb, raised 1 ft in a minute; but as a horse can exert that force but 6 hr a day, one MACHINERY HORSE-POWER is equivalent to that of four horses.

Rules to Determine the Size and Speed of Pulleys or Gears. The driving-pulley is called the DRIVER, and the driven pulley the DRIVEN. If the number of teeth in the gears are used instead of the diameter, in these calculations, number of teeth must be substituted wherever diameter occurs.

(1) **To Find the Diameter of the Driver,** the diameter of the driven and its revolutions, and also revolutions of driver, being given. Multiply the diameter of the driven by its revolutions, and divide the product by the revolutions of the driver; the quotient will give the diameter of the driver.

(2) **To Find the Diameter of the Driven,** the revolutions of the driven, also the diameter and revolutions of the driver, being given. Multiply the diameter of the driver by its revolutions, and divide the product by the revolutions of the driven; the quotient will give the diameter of the driven.

(3) **To Find the Revolutions of the Driver,** the diameter and revolutions of the driven, also the diameter of the driver, being given. Multiply the diameter

* See, also, pages 1274 and 1397.

of the driven by its revolutions, and divide the product by the diameter of the driver; the quotient will give the revolutions of the driver.

(4) **To Find the Revolutions of the Driven,** the diameter and revolutions of the driver, also the diameter of the driven, being given. Multiply the diameter of the driver by its revolutions, and divide the product by the diameter of the driven; the quotient will give the revolutions of the driven.

Horse-Power Transmitted by Belting. The efficiency of belting to transmit power, or to turn a wheel or PULLEY, depends upon the width and thickness of the belt, the arc-contact with the pulley, the position of the belt, whether horizontal, vertical, or at an angle, and the velocity. The greater the velocity and the thicker the belt, the more power it will transmit. A belt running vertically or inclined will transmit less power than one running horizontally, but in figuring the horse-power capacity of belting only the velocity, width and thickness of belt are usually considered, it being assumed that the pulleys are of proper size and located so that the belt will be nearly horizontal. Belts are commonly assumed to be of LEATHER, unless otherwise designated. The term SINGLE BELT is used to designate a belt made of a single thickness of cowhide leather. A DOUBLE BELT is made by cementing and riveting together two thicknesses of leather. There is no standard thickness for either single or double belts.

Rules. Many rules have been given for determining the horse-power that belting will transmit.* Those commonly used are:

(1) For Single Belts. Multiply the width, in inches, by the velocity in feet per minute and divide by 1 000.

(2) For Double Belts. Multiply the width by the velocity and divide by 700. The answer is the number of horse-powers.

Some authorities give divisors of 800 and 733 for single belts, and 550 and 513 for double belts. For the velocity of the belt multiply the number of revolutions per minute of either pulley by the circumference of that pulley.

Notes on Belting. For continuous use a double belt is the most economical in the long run, except on very small pulleys or for very light duty. Triplex and quadruple belts are sometimes used for very heavy duty, but such belts are not commonly carried in stock. Single belts should always be used with the hair-side next the pulley. The belt-speed for maximum economy should be from 4 000 to 4 500 ft per minute. IDLER-PULLEYS work most satisfactorily when located on the slack side of the belt about one-quarter way from the driving-pulley. Belts are more durable and work more satisfactorily when made narrow and thick than when made wide and thin. As belts increase in width they should also be made thicker. For dynamo-work or electric motors the ends of the belt should be fastened together by splicing and cementing instead of by lacing. For all other cases the ends are fastened by hooks or lacing. Belts should be cleaned and greased every 5 to 6 months.

Distance from Center to Center of Shafts.* In locating shafts that are to be connected with each other by belts, care should be taken to separate them by a proper distance. This distance should be such as to allow a gentle sag to the belt when in motion.

Rule. A general rule may be stated thus: Where narrow belts are to be run over small pulleys, 15 ft is a good average, the belt having a sag of from 1½ to 2 in. The minimum distance between shafts is about 10 ft. For larger belts, working on larger pulleys, a distance of from 20 to 25 ft does well, with a sag

* For a discussion of belting, belt-dressing, care of belting, shafting, etc., see Kent's Mechanical Engineers' Pocket-Book.

of from 2½ to 4 in. For main belts, working on very large pulleys, the distance should be from 25 to 30 ft, the belts working well with a sag of from 4 to 5 in. If too great a distance is attempted, the belt will have an unsteady flapping motion, which will destroy both the belt and the machinery.

Arrangement of Belts and Pulleys.* If possible to avoid it, connected shafts should never be placed one directly over the other, as in such case the belt must be kept very tight to do the work. For this purpose belts should be carefully selected of well-stretched leather. It is desirable that the angle of the belt with the floor should not exceed 45°. It is also desirable to locate the shafting and machinery so that belts will run off from each shaft in opposite directions, as this arrangement will relieve the bearings from the friction that would result if all pulled one way on the shaft. If possible, machinery should be so placed that the direction of the belt-motion will be from the top of the driving to the top of the driven pulley, so that the sag will increase the arc of contact. The pulley should be a little wider than the belt required for the work, and should have a crowning face, except where the belt is to be shifted. The motion of driving should run with and not against the laps of the belts.

Rubber Belts are cheaper than leather belts and should always be used in wet places, but for ordinary use in dry places they are not as durable as leather belts. They should always be kept free from grease or animal oils. If they slip, their inside surfaces should be moistened with boiled linseed-oil. Some fine chalk, sprinkled on over the oil, will help the belt.

Rule for Finding the Lengths of Belts. Add the diameter of the two pulleys together, multiply by 3⅐, divide the product by 2, add to the quotient twice the distance between the center of the shafts, and the sum will be the required length.

The Horse-Power that Shafting will Transmit

Diameter of shaft		Revolutions per minute						
		100	150	200	250	300	350	400
in	16th	H.P.	H.P.	H.P.	H.P.	H.P.	H.P.	H.P.
0	15	1.2	1.7	2.4	3.1	3.6	4.3	5.0
1	3	2.4	3.7	4.9	6.1	7.3	8.5	9.7
1	7	4.3	6.4	8.5	10.5	12.7	14.8	16.9
1	11	6.7	10.1	13.4	16.7	20.1	23.4	26.8
1	15	10.0	15.0	20.0	25.0	30.0	35.0	40.0
2	3	14.3	21.4	28.5	35.6	42.7	49.8	57.0
2	7	19.5	29.3	39.0	48.7	58.5	68.2	78.0
2	11	26.0	39.0	52.0	65.0	78.0	87.0	104.0
2	15	33.8	50.6	67.5	84.4	101.3	118.2	135.0
3	3	43.0	64.4	85.8	107.3	128.7	150.3	171.6
3	7	53.6	79.4	107.2	134.0	158.8	187.6	214.4
3	11	65.9	97.9	121.8	164.8	195.7	230.7	263.6
3	15	80.0	120.0	160.0	200.0	240.0	280.0	320.0
4	7	113.9	170.8	227.8	284.7	341.7	398.6	455.6
4	15	156.3	234.4	312.5	390.6	468.7	546.8	625.0

* See Kent's Mechanical Engineers' Pocket-Book.

CHAIN-BLOCKS, HOISTS, HOOKS, ETC.

neral Description. These are portable hoisting-devices which enable one to raise a very heavy load and which sustain the load at any point. In al, they resemble pulleys operated by chains. Since the invention of the ential pulley-block by T. A. Weston, about the year 1863, chain-blocks come into very general use for economical hoisting, particularly where it is ed to hold the load at any point. Chain-blocks are of three general classes:
The Differential Block. This is the original and the simplest and cheap- rm of self-sustaining pulley;
The Screw-Block or Worm-Geared Block. Of these, the Yale & Towne x block is the most efficient type;
The Triplex Block. This is spur-geared.
fferential and worm-geared blocks of all kinds depend upon friction to pre- the load from running down. In the triplex block a separate device is duced which automatically holds the load safely, and yet enables it to be ed with slight effort and at high velocity but without acceleration or danger. is the most efficient of all chain-blocks, and the most economical wherever : work is wanted and economy in time and labor sought. For information the kind of block best adapted to any particular service, the manufacturers ld be consulted. The following data on the power and efficiency of chain- s were supplied by the Yale & Towne Manufacturing Company.

wer and Efficiency of Chain-Hoists. The table below gives the work done by the operator at the hand-pulling chain with each size of the various of chain-blocks in lifting the stated capacity, that is, the amount of work lling required to lift this load ONE FOOT by stating the force exerted in ds and the distance in feet of operating-chains to be pulled. The product se two factors determines the efficiency of the block and the ease and speed isting. The 12, 16, and 20-ton-capacity chain-blocks have each two hand- s.

Work Done by Operator with Chain-Blocks

Capacity, tons	Triplex spur-geared, lb ft	Duplex worm-geared, lb ft	Differential lb ft
½	62× 21	68× 40	122×24
1	82× 31	87× 59	216×30
1½	110× 35	94× 80	246×36
2	120× 42	115× 93	308×42
3	114× 69	132×126	557×38
4	124× 84	142×155	....
5	110×126	145×195	....
6	130×126	145×252	....
8	135×168	160×310	....
10	140×210	160×390	....
12	130×126	....	....
16	135×168	....	....
20	140×210	....	....

e capacities are given in tons. The figures give the number of feet to perated on each hand-chain. A man cannot pull more than his own ht on the operating chains, and can pull faster in proportion as the pull ired is lighter. The maximum pull usually required of one man is 82 lb, he will do more work with less fatigue if the hand-chain pull is not over

40 lb, because he can then pull the chain hand over hand a little more than as fast as he could when pulling twice as hard. When the hand-chain pull is less than 20 lb the speed of hoisting an equal load is diminished, because the man is tired by moving his arms too rapidly, and cannot do as much work as with a heavier pull. The best result is obtained by using a chain-block which has a capacity of double the usual load. The operator then works to the best advantage with average loads, and occasional heavy loads are easily handled without overstraining either the operator or the chain-block, which should never be used beyond its capacity for fear of straining the chain so that it will not run smoothly.

Proportions of Hooks.* For convenience of manufacture hooks of different sizes are made from some regular commercial size of round iron. The basis, or initial point, in each case is, therefore, the size of the iron of which the hook is to be made, and it is indicated by the dimension A in the diagram. The dimension D is arbitrarily assumed. The other dimensions, as given by the formulas, are those which, while preserving a proper bearing face on the interior of the hook for the ropes or chains which may be passed through it, give the greatest resistance to spreading and ultimate rupture which the amount of material in the original bar admits of. The symbol Δ is used in the formulas to indicate the NOMINAL CAPACITY of the hook in tons of 2 000 lb. The formulas which determine the lines of the other parts of the hooks of the several sizes are as follows, all the measurements being expressed in inches:

$D = 0.5\,\Delta + 1.25$	$G = 0.75\,D$
$E = 0.64\,\Delta + 1.60$	$O = 0.363\,\Delta + 0.66$
$F = 0.33\,\Delta + 0.85$	$Q = 0.64\,\Delta + 1.60$
$H = 1.08\,A$	$L = 1.05\,A$
$I = 1.33\,A$	$M = 0.50\,A$
$J = 1.20\,A$	$N = 0.85\,B - 0.16$
$K = 1.13\,A$	$U = 0.866\,A$

Example. To find the dimension D, for a 2-ton hook. The formula is

$$D = 0.5\,\Delta + 1.25$$

and as $\Delta = 2$, the dimension D by the formula is found to be 2¼ in. The dimensions A, are necessarily based upon the ordinary merchant sizes of round iron. The sizes which it has been found best to select are the following:

Capacities of hooks	⅛	¼	½	1	1½	2	3	4	5	6	8	10	tons
Dimension A	⅝	11/16	¾	1 1/16	1¼	1⅜	1¾	2	2¼	2½	2⅞	3¼	inches

The formulas which give the sections of the hook at the several points are all expressed in terms of A, and can therefore be readily ascertained by reference to the foregoing scale.

* By Henry R. Towne, in his Treatise on Cranes, which includes the results of extensive experimental and mathematical investigation.

ɪmple. To find the dimension I, in a 2-ton hook. The formula is

$$I = 1.33\,A$$

or a 2-ton hook, $A = 1\frac{3}{8}$ in. Therefore I, in a 2-ton hook, is found to be in.

anner of Failure of Hooks. Experiment has shown that hooks made ding to the above formulas will give way first by the opening of the jaw, ı, however, will not occur except with a load much in excess of the nominal ity of the hook. This yielding of the hook when overloaded becomes a e of safety, as it constitutes a signal of danger which cannot easily be overd, and which must proceed to a considerable length before rupture occurs the load is dropped. A comparison of these hooks with most of those in ary use shows that the latter are, as a rule, badly proportioned, and fretly dangerously weak.

BELLS

Dimensions and Weights of Church-Bells

Manufactured by Meneely Bell Company, Troy, N. Y.

Bells		Mountings			
Veights, lb	Medium tones	Diameters, in	Sizes of frames, outside, in	Diameters of wheels, ft	in
400	D	27	42×42	4	4
450	C♯	28	42×42	4	4
500	C	29	45×47	4	4
550	C	30	45×47	4	4
600	B	31	45×47	4	9
700	B	33	48×48	5	6
800	B♭	34	48×54	5	6
900	A	36	54×54	5	9
1 000	A	37	54×54	5	9
1 100	A	38	54×59	5	9
1 200	A♭	39	56×59	6	3
1 300	A♭	40	56×59	6	3
1 400	G	41	60×60	6	6
1 500	G	42	60×60	6	6
1 600	G	43	60×60	6	6
1 800	F♯	45	65×68	7	
2 000	F	46	65×68	7	
2 100	F	47	65×68	7	
2 300	E	49	70×72	7	6
2 500	E	50	70×72	7	6
2 800	E♭	51	74×78	8	
3 000	E♭	53	74×78	8	
3 500	D	56	74×78	8	6
4 000	C♯	58	78×81	9	
4 500	C	61	78×81	9	
5 000	C	63	84×84	9	
5 500	B	65	84×84	9	
6 000	B♭	67	84×84	9	6
6 500	B♭	68	90×90	9	6
7 000*	B♭	69	101×90	9	6

A notable example of a 7 000-lb bell is the large bell of the peal in the tower of the opolitan Life Insurance Building, in New York.

e actual weights usually exceed the patterns, noted above, from 2 to 3%.

Meneely School-Bells

Bells		Mountings
Weights, lb	Diameters, in	Sizes of frames, outside, ft in ft in
100	17	2 6 × 2 8
125	18½	2 6 × 2 8
150	19½	2 6 × 2 8
200	21½	2 8 × 3 0
250	23	3 0 × 3 2
300	24½	3 0 × 3 4
350	26	3 0 × 3 4

Sizes of Rope for Bells

	Diameter, in
For bells of less than 500 lb	½
For bells of 500 to 800 lb	⅝
For bells of 800 to 1 800 lb	¾
For bells above 1 800 lb	⅞ to 1

The Largest Bells in the World *

Names and locations of bells	Date cast	Actual vibration	Keynote	Diameter, in	Sound-bow Inches	Sound-bow Stroke	[illegible]
Moscow, Tzar Kolokol †	1733	74	D	272	23	0.84	[illegible]
Burmah, Mingoon	....	94	F♯	203?	16?	0.80	[illegible]
Moscow, St. Ivan's	1819	105	G♯	185	14.75	0.80	[illegible]
Pekin, Great Bell	....	......	....	156	......	......	[illegible]
Burmah, Maha Ganda	....	125	B	155	12.5	0.80	[illegible]
Nishni Novgorod	....	125	B	151	12	0.80	[illegible]
Moscow, Church of Redeemer	1879	141	C♯	136.3?	10.6	0.80	[illegible]
Nankin, China	....	......	....	112	......	......	[illegible]
London, St. Paul's	1881	157	E♭	114.25	8.75	0.76	[illegible]
Olmutz, Bohemia	....	157	E♭	121	9.125	0.75	[illegible]
Vienna, Austria	1711	157	E♭	118	9.5	0.80	[illegible]
Westminster, London	1856	166	E	113.5	9.375	0.83	[illegible]
Erfurt, Saxony	1487	176	F	103.6	9.75	0.75	[illegible]
Notre Dame, Paris	1680	166	E	103	7.5	0.73	[illegible]
Montreal, Canada	1847	176	F	103	7.8	0.76	[illegible]
York, England	1845	187	F♯	100	8	0.80	[illegible]
St. Peter's, Rome	1786	187	F♯	97.25	7.5	0.77	[illegible]
Great Tom, Oxford	1680	210	G♯	84	6.125	0.73	[illegible]
Cologne, Germany	1477	198	G	95	7.2	0.76	[illegible]
Brussels, Belgium	....	210	G♯	95.81	7.75	0.71	[illegible]
State-house, Philadelphia	1875	198	G	88	6.375	0.73	[illegible]
Lincoln, England	1834	210	G♯	82.85	6	0.73	[illegible]
St. Paul's, London	1716	222	A	81	6.08	0.75	[illegible]
Exeter, England	1675	210	G♯	76	5	0.66	[illegible]
Old Lincoln, England	1610	249	B	75.5	5.94	0.78	[illegible]
Westminster, London	1857	249	B	72	5.75	0.79	[illegible]

* John W. Nystrom, in the Journal of the Franklin Institute, Philadelphia.

† This bell is fractured and has not been rung for many years.

SYMBOLS FOR THE APOSTLES AND SAINTS

om the constant occurrence of symbols in the edifices of the Middle Ages nany of the cathedrals of the present day, the following list of symbols, as nonly attached to the apostles and saints, may be found useful:

Holy Apostles

eter. Bears a key, or two keys with different wards.
ndrew. Leans on a cross so called from him; called by heralds the saltire.
ohn the Evangelist. With a chalice, in which is a winged serpent. When this symbol is used, the eagle, another symbol of him, is never given.
artholomew. With a flaying-knife.
ames the Less. A fuller's staff bearing a small square banner.
ames the Greater. A pilgrim's staff, hat and escalop-shell.
homas. An arrow, or with a long staff.
imon. A long saw.
ude. A club.
Iatthias. A hatchet.
hilip. Leans on a spear or has a long cross in the shape of a T.
Iatthew. A knife or dagger.
Iark. A winged lion.
uke. A bull.
ohn. An eagle.
aul. An elevated sword, or two swords in saltire.
ohn the Baptist. An Agnus Dei.
tephen. With stones in his lap.

Saints

gnes. A lamb at her feet.
ecilia. With an organ.
lement. With an anchor.
avid. Preaching on a hill.
enis. With his head in his hands.
eorge. With the dragon.
licholas. With three naked children in a tub, in the end whereof rests his pastoral staff.
incent. On the rack.

IRCULAR OF ADVICE RELATIVE TO PRINCIPLES OF PROFESSIONAL PRACTICE AND THE CANONS OF ETHICS, BY THE AMERICAN INSTITUTE OF ARCHITECTS *

A Circular of Advice

troductory. The American Institute of Architects, seeking to maintain a standard of practice and conduct on the part of its members as a safeguard e important financial, technical and esthetic interests entrusted to them, s the following advice relative to professional practice: The profession of itecture calls for men of the highest integrity, business capacity and artistic

* The American Institute of Architects, Document No. 141, Washington, D. C., 29, 1919. Reprinted by permission. This circular relates to the principles of sional practice and the canons of ethics.

ability. The architect is entrusted with financial undertakings in which honesty of purpose must be above suspicion; he acts as professional adviser to his client and his advice must be absolutely disinterested; he is charged with the exercise of judicial functions as between client and contractors and must act with entire impartiality; he has moral responsibilities to his professional associates and subordinates; finally, he is engaged in a profession which carries with it grave responsibility to the public. These duties and responsibilities cannot be properly discharged unless his motives, conduct and ability are such as to command respect and confidence. No set of rules can be framed which will particularize all the duties of the architect in his various relations to his clients, to contractors, to his professional brethren, and to the public. The following principles should, however, govern the conduct of members of the profession and should serve as a guide in circumstances other than those enumerated:

(1) **On the Architect's Status.** The architect's relation to his client is primarily that of PROFESSIONAL ADVISER; this relation continues throughout the entire course of his service. When, however, a contract has been executed between his client and a contractor by the terms of which the architect becomes the official interpreter of its conditions and the judge of its performance, an additional relation is created under which it is incumbent upon the architect to side neither with client nor contractor, but to use his powers under the contract to enforce its faithful performance by both parties. The fact that the architect's payment comes from the client does not invalidate his obligation to act with impartiality to both parties.

(2) **On Preliminary Drawings and Estimates.** The architect at the outset should impress upon the client the importance of sufficient time for the preparation of drawings and specifications. It is the duty of the architect to make or secure PRELIMINARY ESTIMATES when requested, but he should acquaint the client with their conditional character and inform him that complete and final figures can be had only from complete and final drawings and specifications. If an unconditional limit of cost be imposed before such drawings are made and estimated, the architect must be free to make such adjustments as seem to him necessary. Since the architect should assume no responsibility that may prevent him from giving his client disinterested advice, he should not, by bond or otherwise, guarantee any estimate or contract.

(3) **On Superintendence and Expert Services.** On all work except the simplest, it is to the interest of the owner to employ a SUPERINTENDENT or CLERK OF THE WORKS. In many engineering problems and in certain specialized problems, it is to his interest to have the services of special experts and the architect should so inform him. The experience and special knowledge of the architect make it to the advantage of the owner that these persons, although paid by the owner, should be selected by the architect under whose direction they are to work.

(4) **On the Architect's Charges.** The SCHEDULE OF CHARGES of the American Institute of Architects is recognized as a proper minimum of payment. The locality or the nature of the work, the quality of services to be rendered, the skill of the practitioner or other circumstances frequently justify a higher charge than that indicated by the schedule.

(5) **On Payment for Expert Service.** The architect when retained as an EXPERT, whether in connection with competitions or otherwise, should receive a compensation proportionate to the responsibility and difficulty of the service. No duty of the architect is more exacting than such service, and the

ofession is involved in it. Under no circumstances should experts know-
name prices in competition with each other.

On Selection of Bidders or Contractors. The architect should advise ent in the selection of BIDDERS and in the AWARD OF THE CONTRACT. In ng that none but trustworthy bidders be invited and that the award be only to contractors who are reliable and competent, the architect protects erests of his client.

On Duties to the Contractor. As the architect decides whether or e intent of his plans and specifications is properly carried out, he should pecial care to see that these drawings and specifications are complete and te, and he should never call upon the contractor to make good oversights rs in them nor attempt to shirk responsibility by indefinite clauses in the ct or specifications.

On Engaging in the Building Trades. The architect should not directly irectly engage in any of the BUILDING TRADES. If he has any financial t in any building material or device, he should not specify or use it without owledge and approval of his client.

On Accepting Commissions or Favors. The architect should not any COMMISSION or any substantial service from a contractor or from terested person other than his client.

On Encouraging Good Workmanship. The large powers with which chitect is invested should be used with judgment. While he must con- bad work, he should commend good work. Intelligent initiative on the craftsmen and workmen should be recognized and encouraged and the ct should make evident his appreciation of the dignity of the ARTISAN'S ION.

On Offering Services Gratuitously. The seeking out of a possible and the offering to him of professional services on approval and WITHOUT NSATION, unless warranted by personal or previous business relations, o lower the dignity and standing of the profession and is to be condemned.

On Advertising. Publicity of the standards, aims and progress of the ion, both in general and as exemplified by individual achievement, is al. Advertising of the individual, meaning self-laudatory publicity ed by the person advertised or with his consent, tends to defeat its own to the individual as well as to lower the dignity of the profession, and is leplored.

On Signing Buildings and Use of Titles. The unobtrusive SIGNATURE LDINGS after completion is desirable. The placing of the ARCHITECT'S ON A BUILDING DURING CONSTRUCTION serves a legitimate purpose for information, but it is to be deplored if done obtrusively for individual ty. The use of INITIALS designating membership in the Institute is le in all professional relationships, in order to promote a general under- g of the Institute and its standards through a knowledge of its members eir professional activities. Upon the members devolves the responsibility ciate the symbols of the Institute with acts representative of the highest rds of professional practice.

On Competitions. An architect should not take part in a competition MPETITOR or JUROR unless the competition is to be conducted according best practice and usage of the profession, as evidenced by its having d the approval of the Institute, nor should he continue to act as PRO- NAL ADVISER after it has been determined that the program cannot be so

drawn as to receive such approval. When an architect has been auth submit sketches for a given project, no other architect should submit for it until the owner has taken definite action on the first sketches, far as the second architect is concerned, a competition is thus esta Except as an authorized competitor, an architect may not attempt to work for which a competition has been instituted. He may not atte influence the award in a competition in which he has submitted drawings may not accept the commission to do the work for which a competition has instituted if he has acted in an advisory capacity either in drawing the pro or in making the award.

(15) **On Injuring Others.** An architect should not falsely or mali injure, directly or indirectly, the professional reputation, prospects or bu of a fellow architect.

(16) **On Undertaking the Work of Others.** An architect should not take a commission while the claim for compensation or damages or b an architect previously employed and whose employment has been term remains unsatisfied, unless such claim has been referred to arbitration or has been joined at law; or unless the architect previously employed ne to press his claim legally; nor should he attempt to supplant a fellow arc after definite steps have been taken toward his employment.

(17) **On Duties to Students and Draughtsmen.** The architect s advise and assist those who intend making architecture their career. I beginner must get his training solely in the office of an architect, the latter s assist him to the best of his ability by instruction and advice. An arc should urge his draughtsmen to avail themselves of educational opportun He should, as far as practicable, give encouragement to all worthy agencie institutions for architectural education. While a thorough technical pr ation is essential for the practice of architecture, architects cannot too str insist that it should rest upon a broad foundation of general culture.

(18) **On Duties to the Public and to Building Authorities.** An a tect should be mindful of the public welfare and should participate in movements for public betterment in which his special training and exp quality him to act. He should not, even under his client's instructions, in or encourage any practices contrary to law or hostile to the public in for as he is not obliged to accept a given piece of work, he cannot, by urg he has but followed his client's instructions, escape the condemnation at to his acts. An architect should support all public officials who have of building in the rightful performance of their legal duties. He shoul fully comply with all building laws and regulations, and if any such ap him unwise or unfair, he should endeavor to have them altered.

(19) **On Professional Qualifications.** The public has the right to that he who bears the TITLE OF ARCHITECT has the knowledge and ability for the proper invention, illustration and supervision of all building op which he may undertake. Such qualifications alone justify the assum the title of architect.

The Canons of Ethics *

The following Canons are Adopted by The American Insti Architects as a general guide, yet the enumeration of particular duties

* Adopted, December 14–16, 1909. Revised, December 10–12, 1912. Revis 26, 1918.

ot be construed as a denial of the existence of others equally important although ot specially mentioned. It should also be noted that the several sections dicate offenses of greatly varying degrees of gravity. It is unprofessional for architect

(1) To engage directly or indirectly in any of the building or decorative trades.

(2) To guarantee an estimate or contract by bond or otherwise.

(3) To accept any commission or substantial service from a contractor or om any interested party other than the owner.

(4) To take part in any competition which has not received the approval of e Institute or to continue to act as professional adviser after it has been dermined that the program cannot be so drawn as to receive such approval.

(5) To attempt in any way, except as a duly authorized competitor, to cure work for which a competition is in progress.

(6) To attempt to influence, either directly or indirectly, the award of a comtition in which he is a competitor.

(7) To accept the commission to do the work for which a competition has been stituted if he has acted in an advisory capacity, either in drawing the proam or in making the award.

(8) To injure falsely or maliciously, directly or indirectly, the professional putation, prospects, or business of a fellow architect.

(9) To undertake a commission while the claim for compensation, or damages, both, of an architect previously employed and whose employment has been rminated remains unsatisfied, until such claim has been referred to arbitration issue has been joined at law, or unless the architect previously employed glects to press his claim legally.

(10) To attempt to supplant a fellow architect after definite steps have been ken toward his employment, that is, by submitting sketches for a project for ich another architect has been authorized to submit sketches.

(11) To compete knowingly with a fellow architect for employment on the sis of professional charges.

ofessional Practice of Architects. Details of Service to be Rendered and Schedule of Proper Minimum Charges *

(1) The architect's professional services consist of the necessary conferences, e preparation of preliminary studies, working drawings, specifications, largele and full-size detail drawings; the drafting of forms of proposals and ntracts; the issuance of certificates of payment; the keeping of accounts, the neral administration of the business and supervision of the work, for which, cept as hereinafter mentioned, the minimum charge, based upon the total t of the work † complete, is 6 per cent.

(2) On residential work, alterations to existing buildings, monuments, furnie, decorative and cabinetwork and landscape-architecture, it is proper to ke a higher charge than above indicated.

* As adopted at the Washington, D. C., Convention, December 15–17, 1908, and revised in form at the Minneapolis convention, December 6–8, 1916.

† The words "the cost of the work," as used in Articles (1) and (9) hereof, are ordinarily be interpreted as meaning the total of the contract-sums incurred for the execution of work, not including architect's and engineer's fees or the salary of the clerk of the ks, but in certain rare cases, that is, when labor or material is furnished by the owner w its market cost or when old materials are reused, the cost of the work is to be erpreted as the cost of all materials and labor necessary to complete the work, as h cost would have been if all materials had been new and if all labor had been fully d at market prices current when the work was ordered, plus contractor's profits and enses

(3) The architect is entitled to compensation for articles purchased u his direction, even though not designed by him.

(4) Where the architect is not otherwise retained, consultation-fees for pro fessional advice are to be paid in proportion to the importance of the quest involved and services rendered.

(5) The architect is to be reimbursed for the costs of transportation a living incurred by him and his assistants while traveling in discharge of du connected with the work, and the costs of the services of heating, ventilati mechanical, and electrical engineers.

(6) The rate of percentage arising from Articles (1) and (2) hereof, that the basic rate, applies when all of the work is let under one contract. Shoul the owner determine to have certain portions of the work executed und separate contracts, as the architect's burden of service, expense, and respon bility is thereby increased, the rate in connection with such portions of the wo is greater (usually by 4 per cent) than the basic rate. Should the owner dete mine to have substantially the entire work executed under separate contrac then such higher rate applies to the entire work. In any event, however, t basic rate, without increase, applies to contracts for any portions of the wor on which the owner reimburses the engineer's fees to the architect.

(7) If, after a definite scheme has been approved, the owner makes a decisi which, for its proper execution, involves extra services and expense for chang in or additions to the drawings, specifications, or other documents; or if a con tract be let by cost of labor and materials plus a percentage or fixed sum; or i the architect be put to labor or expense by delays caused by the owner or a con tractor, or by the delinquency or insolvency of either, or as a result of damage b fire, he is to be equitably paid for such extra service and expense.

(8) Should the execution of any work designed or specified by the archite or any part of such work be abandoned or suspended, the architect is to be pai in accordance with or in proportion to the terms of Article (9) of this Schedu for the service rendered on account of it, up to the time of such abandonment o suspension.

(9) Whether the work be executed or whether its execution be suspended o abandoned in part or whole, payments to the architect on his fee are subje to the provisions of Articles (7) and (8), made as follows: Upon completi of the preliminary studies, a sum equal to 20 per cent of the basic rate compu upon a reasonable estimated cost. Upon completion of specifications and ge eral working drawings (exclusive of details) a sum sufficient to increase pa ments on the fee to sixty per cent of the rate or rates of commission agreed up as influenced by Article (6), computed upon a reasonable cost estimated o such completed specifications and drawings, or if bids have been receiv then computed upon the lowest bona-fide bid or bids. From time to tim during the execution of work and in proportion to the amount of service re dered by the architect, payments are made until the aggregate of all paymen made on account of the fee under this Article reaches a sum equal to the ra or rates of commission agreed upon as influenced by Article (6), computed up the final cost of the work. Payments to the architect, other than those on h fee, fall due from time to time as his work is done or as costs are incurred. N deduction is made from the architect's fee on account of penalty, liquidat damages or other sums withheld from payments to contractors.

(10) The owner is to furnish the architect with a complete and accura survey of the building-site, giving the grades and lines of streets, paveme and adjoining properties; the rights, restrictions, easements, boundaries a contours of the building-site, and full information as to sewer, water, gas a

ctrical service. The owner is to pay for borings or test-pits and for chemical echanical or other tests, when required.

(11) The architect endeavors to guard the owner against defects and deiencies in the work of contractors, but he does not guarantee the performance their contracts. The supervision of an architect is to be distinguished from continuous personal superintendence to be obtained by the employment a clerk of the works. When authorized by the owner, a clerk of the works, eptable to both owner and architect, is to be engaged by the architect at a ary satisfactory to the owner and paid by the owner, upon presentation of architect's monthly certificates.

(12) When requested to do so, the architect makes or procures preliminary imates on the cost of the work and he endeavors to keep the actual cost of work as low as may be consistent with the purpose of the building and with per workmanship and material, but no such estimate can be regarded as er than an approximation.

(13) Drawings and specifications, as instruments of service, are the property the architect, whether the work for which they are made be executed or not.

ARCHITECTURAL COMPETITIONS *

This Circular of Advice furnishes information as to the best methods of iducting architectural competitions and states the conditions which are prequisite to participation in them by members of The American Institute of chitects. It does not apply to competitions for work to be erected elsewhere n in the United States, its territories and possessions.

The Attitude of The American Institute of Architects to Competitions. ce its foundation, more than sixty years ago (1857), The American Institute Architects has given much attention to the conduct of ARCHITECTURAL COMPITIONS. These contests, instituted when the direct selection of an architect ld not be made, were for many years conducted without proper regulation l often in disregard of the interests both of the owner and of the competitors. e owner, totally unfamiliar with the intricacies of the subject, assumed, with- skilled assistance, to prepare the programme, laying down, or more frequently oring, rules to govern procedure. With the growth of the country, the inase in expenditures for public and private buildings, and the increase in the nber of architects, all the evils of ill-regulated competitions became more rked. Programmes varied from loose and careless forms, difficult to undernd and often open to the suspicion that only the initiated knew what they ant, to over-elaborate ones necessitating useless study of details and needless wings. Those instituting the competition often had no legal authority to pay competitors, still less to employ the winner. There was great economic ste, the total cost of participation exceeding the total net profit accruing to the fession from work secured through competitions. Architects have learned t the outcome of a competition, unless governed by well-defined agreements, rgely a matter of chance. The owner has, to be sure, a choice of designs, but

*The American Institute of Architects, Document 114. Reprinted by permission. horized by the 43d annual convention at Washington, D. C., December 14-16, 1909; ed March 30, 1910, amended June 10, 1910; and January 3, 1911; ratified by the annual convention at San Francisco, January 16-21, 1911; reaffirmed by the 45th ual convention at Washington, D. C.; amended January 3, 1912, as authorized by convention; amended December 9, 1912, and ratified by the 46th annual convention Vashington, D. C., December 10-12, 1912; amended December 2, 1913, and ratified the 47th annual convention at New Orleans, La., December 3-5, 1913; amended and ied by the 48th annual convention at Washington, D. C., December 2-4, 1914.

he is no more likely to make the wisest selection or to obtain the best buil-
than if he selects his architect directly, guided by the results previously achi-
by the men he is considering. When a competition is necessary or desir-
it should be of such form as to establish equitable relations between the ow-
and the competitors. To insure this:

(1) The REQUIREMENTS should be clear and definite, and the statement of
them, since it must be in technical terms, should be drawn by one familiar wi-
such terms.

(2) The COMPETENCY of all competing should be assured. The drawings sub-
mitted in a competition are evidence, only in part, of the ability of the archit-
to execute the building. The owner, for his own protection, should admit to
the competition only those to whom he would be willing to entrust the wo-
that is, to men of known honesty and competence.

(3) The AGREEMENT between the owner and the competitors should be
definite, as becomes a plain statement of business relations.

(4) The JUDGMENT should be based on knowledge, and since ideas present-
in the form of drawings are intelligible only to a trained mind, judgment sho-
not be rendered until the owner has received competent technical advice as to
the merits of those ideas.

To sum up: To insure the best results, a competition should have (1) a clea-
programme, (2) competent competitors, (3) a business agreement, (4) a fa-
judgment.

Fifteen years ago many competitions had none of these provisions and fe-
had all of them. The commonest form of competition was one that wa-
open to all, had a programme prepared by a layman, was judged by the owne-
without professional assistance, contained no agreement, and made no provisi-
to eliminate the incompetent. All this demanded correction. The Institute,
seeking a means of reform, perceived at once that its relation to the owner cou-
be only an ADVISORY one. It might advise him how to hold a competition,
but it could go no further. To architects in general the Institute cou-
scarcely presume to offer even its advice, but being a professional bod-
charged with maintaining ethical standards among its own members, its du-
was to see that they did not take part in competitions that fell below a reaso-
able standard.

It was, therefore, voted in convention of the Institute that members sho-
be free to take part in competitions only when their terms had received th-
APPROVAL OF THE INSTITUTE. Thereupon the Institute fully stated the prin-
ciples which should govern competitions and defined the conditions prerequisi-
to the giving of its approval. These are contained in the CIRCULAR OF ADVI-
here following, which is intended as a guide to all who are interested in com-
petitions. Committees of the Institute throughout the country are authoriz-
to give its approval to competitions when properly conducted, but unless t-
programme has received such approval members of the Institute do not accep-
a position as competitor or juror, nor does a member continue to act as profe-
sional adviser after it becomes evident that the owner will not permit hi-
programme to be brought into harmony with the principles approved by th-
Institute.

The position thus taken by the Institute is by no means an arbitrary one, si-
it governs the action of none but its own members. To the owner its servi-
has been of great value in giving him information and useful advice and in savi-
him from the delays, cost and disappointment incident to the amateur cond-
of a competition. The owner who disregards the standard set by the Instit-
finds it increasingly difficult to get men of standing in the profession to ent-
He who raises his programme to that standard has no difficulty in securing th-

vices of architects of the greatest ability. Even in the few years since the titute first made its firm stand against the abuses of competitions, the effect that action has been far greater than could have been foreseen. It has not agether eliminated ill-regulated competitions, but it has greatly reduced their nber, and it is safe to say that no competition of prime importance is now ducted except in accordance with the principles stated in the following CIR-LAR OF ADVICE:

Circular of Advice and Information Relative to the Conduct of Architectural Competitions

Competitions are instituted to enable the OWNER * to choose an ARCHITECT ough comparison of the designs submitted. The American Institute of Archi-ts, believing that the interests of both owner and competitors are best served fair and equitable agreements between them, issues this circular as a STATE-NT OF THE PRINCIPLES which should underlie such agreements. The Institute s not assume to dictate the owner's course in conducting competitions, but s to assist him by advising the adoption of such methods as experience has ved to be just and wise. So important, however, does the adoption of such thods appear to architects that members of the Institute do not take part in npetitions except under conditions based on this circular and specifically set h in Articles (16) and (18).

1) **On Competitions in General.** A competition exists when two or more itects prepare sketches at the same time for the same project, but no archi- who prepares drawings for comparison in problems of an altruistic or edu-onal nature, where the problem does not involve a definite proposed building ration, shall be held as having taken part in a competition, within the mean- of this circular of advice.

2) **On the Employment of a Professional Adviser.** No competition l be instituted without the aid of a competent ADVISER. He should be an itect of the highest standing and his selection should be the owner's first . He must be chosen with the greatest care, as the success of the competition depend largely upon his experience and ability. The EXPERT'S ADVICE is reat value to the owner, for example, in so drawing the programme as to guard him against the employment of an architect who submits a design ely exceeding in cost of execution the sum at his disposal, and in helping him void the disappointment, embarrassment and litigation which so often result n competitions conducted without expert technical advice. The DUTIES THE EXPERT are to advise those who hold the competition as to its form and ns, to draw up the programme, to advise in choosing the competitors, to wer their questions, and to conduct the competition.

3) **On the Forms of Competition.** The following forms of competition recognized:

LIMITED. In this form, participation is limited to a certain number of archi-s whose names should be stated in the programme and to any one of whom owner is willing to entrust the work. In a LIMITED COMPETITION the com-itors may be chosen (a) from among architects whose ability is so evident t no formal inquiry into their qualifications is needed, or (b) from among hitects who make application accompanied by evidence of their education experience. The limited form has the advantage that the owner and the fessional adviser may meet competitors and discuss the terms of the com-

* The person, corporation or other entity instituting a competition, whether acting ctly or through representatives, is herein called the OWNER.

petition with them before the issuance of the programme Form (a) is simplest and most direct form of competition.

OPEN. The Institute believes that a competition OPEN TO ALL who wish participate without regard to their qualifications is detrimental to the interests alike of owner and of architects. It will, therefore, give its approval to that form only when conducted in two stages, since by that means alone it is possible to insure anonymity of submission while safeguarding the owner's interests against the selection as winner of a person lacking the qualifications set forth in Article (4) hereof. In this form there is a FIRST STAGE open to all, in which the competitive drawings are of the slightest nature, involving only the fundamental ideas of the solution. These drawings are accompanied by evidence of the competitor's education and experience. From the first stage a small number who have thus demonstrated their competence to design the work and to carry it successfully into execution are chosen to take part in A FINAL and strictly ANONYMOUS STAGE involving competitive drawings of the type indicated in Article hereof.

(4) **On the Qualification of Competitors.** The interests of the owner may be seriously prejudiced by admitting to a limited competition or to the second stage of an open competition any architect who has not established to the satisfaction of the owner his competence to design and execute the work. It is sometimes urged that by admitting all who wish to take part some unknown but brilliant designer may be found. If the object of a competition were a set of sketches, such reasoning might be valid. But sketches give no evidence that their author has the matured artistic ability to fulfil their promise, or that he has the technical knowledge necessary to control the design of the highly complex structure and equipment of a modern building, or that he has executive ability for large affairs, or the force to compel the proper execution of contracts. Attempts have often been made to defend the owner's interests by associating an architect of ability with one lacking in experience. These have generally resulted in failure. As the owner should feel bound, not only legally, but in point of honor, to retain as his architect the competitor to whom the award is made, it is essential that the competitors in a limited competition, or in the second stage of an open competition, should be selected with the greatest care in consultation with the professional adviser, and that there should be included among them only architects in whose ability and integrity the owner has absolute confidence, and to any one of whom he is willing to entrust the work.

(5) **On the Number of Competitors.** Experience has demonstrated that the admission of MANY COMPETITORS is detrimental to the success of a competition. When there are many, each knows that he has but a slight chance of success, and he is therefore less aroused to his best effort than when there are a few. As the owner is interested only in the best result, he is ill-advised to sacrifice quality for quantity.

(6) **On Anonymity of Competitors.** Absolute and effective ANONYMITY is a necessary condition of a fair and unbiased competition. The SIGNING OF DRAWINGS should not be permitted nor should they bear any motto, device or distinguishing mark. Drawings and the accompanying sealed envelopes containing their authors' names should be numbered upon receipt, the envelopes remaining unopened until after the award.

(7) **On the Cost of the Proposed Work.** No statement of the intended COST OF THE WORK should be made unless it has been ascertained that the work as described in the programme can be properly executed within the sum named. In ge[illegible] wiser to limit the cubic contents of the building than to state

t of cost. The programme should neither require nor permit competitors urnish their own or builders' estimates of the cost of executing the work in ordance with their designs. Such estimates are singularly unreliable. If cubage be properly limited they are unnecessary.

8) **On the Jury of Award.** To insure a wise and just award and to pro- the interests of both the owner and the competitors, the competitive draw- should be submitted to a JURY so chosen as to secure expert knowledge and dom from personal bias. Such a jury thoroughly understands and can lain the intent of the drawings. It discovers from them their authors' l in design, arrangement and construction. Because of its trained judgment advice as to the merits of the designs submitted is of the highest value to the ner. The jury must consist of at least three members, one of whom must, a majority of whom should, be PRACTICING ARCHITECTS. One or more mbers of the jury may be chosen by the competitors. It is the DUTY OF THE Y to study carefully the programme and all conditions relating to the problem the competition before examining the designs submitted; to refuse to make ecommend an award in favor of the author of any design that does not fulfil conditions distinctly stated as mandatory in the programme; to give ample e to the careful study of the designs; and to render a decision only after ure consideration. The jury should see to it that a copy of its report reaches ry competitor. The professional adviser should not be a member of the , as his judgment is apt to be influenced by his previous study of the problem.

9) **On the Competitive Drawings.** The purpose of an architectural com- tion is not to secure fully developed plans, but such evidence of skill in treat- the essential elements of the problem as will assist in the SELECTION OF AN HITECT. The drawings should, therefore, be as few in number and as simple haracter as will express the general design of the building. A jury of experts s not need elaborate drawings.

10) **On the Programme.** The programme should contain rules for the duct of the competition, instructions for competitors and the jury, and the ement between the owner and the competitors. Uniform conditions for all petitors are fundamental to the proper conduct of competitions. Lengthy grammes and detailed instructions as to the desired accommodations should avoided as they confuse the problem and hamper the competitors. The blem should be stated broadly. Its solutions should be left to the competi- . A distinction should be clearly drawn between the MANDATORY and the ISORY provisions of the programme, that is, between those which, if not met, clude an award in favor of the author of a design so failing, and those which merely optional or of a suggestive character. The mandatory requirements uld be set forth in such a way that they cannot fail to be recognized as such. y should be as few as possible, and should relate only to matters which not be left to the discretion of the competitors. It is difficult to summarize fly the programme, but it should at least:

a) Name the owner of the structure forming the subject of the competition, l state whether the owner institutes the competition personally or through resentatives; if the latter, name the representatives, state how their author- is derived, and define its scope.

b) State the kind of competition to be instituted, and in limited competitions ne the competitors; or in open competitions, if the competition is limited graphically or otherwise, state the limits.

c) Fix a time and place for the receipt of the designs. The time should not altered except with the unanimous consent of the competitors.

d) Furnish exact information as to the site.

(e) State the desired accommodation, avoiding detail.
(f) State the cost if it be fixed or, better, limit the cubic contents.
(g) Fix uniform requirements for the drawings, giving the number, the s or scales, and the method of rendering.
(h) Forbid the submission of more than one design by any one competito
(i) Provide a method for insuring anonymity of submission.
(j) Name the members of the jury or provide for their selection. De their powers and duties. If for legal reasons the jury may not make the award, state such reasons and in whom such power is vested.
(k) Provide that no award shall be made in favor of any design until the j shall have certified that it does not violate any mandatory requirement of programme.
(l) Provide that during the competition there shall be no communicati relative to it between any competitor and the owner, his representatives, or member of the jury, and that any communication with the professional advi shall be in writing. Provide also that any information, whether in answer such communications or not, shall be given in writing simultaneously to competitors. Set a date after which no questions will be answered.
(m) State the number and amount of payments to competitors.
(n) Provide that the professional adviser shall send a report of the compe tion to each competitor, including therein the report of the jury.
(o) Provide that no drawing shall be exhibited or made public until after award of the jury.
(p) Provide for the return of unsuccessful drawings to their respective auth within a reasonable time.
(q) Provide that nothing original in any of the unsuccessful designs shall used without consent of, and compensation to, the author of the design in wh it appears.
(r) Include the contract between the owner and the competitors.
(s) Include the contract between the owner and the architect receiving award.

(11) **On the Agreement.** An owner who institutes a competition assu a moral obligation to retain one of the competitors as his architect. In o that architects invited to compete may determine whether they will take p it is essential that they should know the terms upon which the winner wil employed; and it is of the utmost importance to the owner that those te should be so clearly defined that no disagreement as to their meaning can after the award is made. Unless they be so defined, delay is likely to occur disagreements to arise at a time when a complete understanding between ow and architect is most important for the welfare of the work. Therefore, th must be included in the programme a form which guarantees the appointm of one of the competitors as architect and provides an agreement operat upon that appointment, defining his employment in terms consonant with best practice. This must conform in all fundamental respects to the ty form of agreement appended to this circular.

(12) **On Payments to Unsuccessful Competitors.** In a limited co petition and in the second stage of an open competition each competit except the winner, should be paid for his services.

(13) **On Legality of Procedure.** It is highly important that each s taken in connection with a competition and every provision of the progra should be in consonance with law. Those charged with holding the competit should know and state their authority. If they are not empowered to bind th principal by contracts with the competitors, they should seek and receive

authority before issuing an invitation. If authority cannot legally be granted to the jury to make the award, that fact should be stated, and the body named in which such authority is vested.

(14) **On the Conduct of the Owner.** In order to maintain absolute impartiality toward all competitors, the owner, his representatives and all connected with the enterprise should, as soon as a professional adviser has been appointed, refrain from holding any communication in regard to the matter with any architect except the adviser or the jurors. The meeting with competitors described in Article (3) is of course an exception.

(15) **On the Conduct of Architects.** An architect should not attempt in any way, except as a duly authorized competitor, to secure work for which a competition is in progress, nor should he attempt to influence, either directly or indirectly, the award in a competition in which he is a competitor. An architect should not accept the commission to do the work for which a competition has been instituted if he has acted in an advisory capacity, either in drawing the programme or making the award. An architect should not submit in competition a design which has not been produced in his own office or under his own direction. No competitor should enter into association with another architect, except with the consent of the owner. If such associates should win the competition, their association should continue until the completion of the work thus won. During the competition, no competitor should hold any communication relative to it with the owner, his representatives or any member of the jury, nor should he hold any communication with the professional adviser, except it be in writing. When an architect has been authorized to submit sketches for a given project, no other architect should submit sketches for it until the owner has taken definite action on the first sketches, since, as far as the second architect is concerned, a competition is thus established.

(16) **On the Participation of Members of the Institute.** Members of The American Institute of Architects do not take part as competitors or jurors in any competition the programme of which has not received the formal approval of the Institute, nor does a member continue to act as professional adviser after it has been determined that the programme cannot be so drawn as to receive such approval.

(17) **Committees.** In order that the advice of the Institute may be given to those who seek it and that its approval may be given to programmes in consonance with its principles, the Institute maintains the following committees:

(a) The STANDING COMMITTEE ON COMPETITIONS, representing the Institute in its relation to competitions generally. This committee advises the subcommittees and directs their work and they report to it.

(b) A SUBCOMMITTEE for the territory of each chapter, representing the Institute in its relation to competitions for work to be erected within such territory.

The president of the chapter is EX-OFFICIO chairman of the subcommittee, the other members of which he appoints. The subcommittees derive their authority from the Institute and not from the chapters. An appeal from the decision of a subcommittee may be made to the standing committee. The standing committee may approve, modify or annul the decision of a subcommittee.

(18) **The Institute's Approval of the Programme.** The approval of the Institute is not given to a programme unless it meets the following essential conditions:

(a) That there be a professional adviser.

(b) That the competition be of one of the forms described in Article (3).

(c) That the programme contain an Agreement and Conditions of Contract between architect and owner in conformity with those printed in the Appendix of this circular, that it include no provision at variance therewith, that it contain terms of payments in accord with good practice, and that it specifically set forth the nature of expert engineering services for which the architect will be reimbursed.

(d) That the programme make provision for a jury of at least three persons.

(e) That the programme conform in all particulars to the spirit of this circular.

When the programme meets the above essential conditions, the approval of the Institute may be given to it by the subcommittee for the territory in which the work is to be erected, or if there be no subcommittee for that territory, then by the standing committee on competitions. If, for legal or other reasons, the standing committee deem that deviations from the essential conditions are justified, it may give the approval of the Institute to a programme containing such deviations. Power to give approval in such cases is, however, vested only in the standing committee. The professional adviser, when duly authorized in writing by the proper committee, may print the Institute's approval as a part of the programme or otherwise communicate it to those invited to compete.

Typical Form of Agreement between Owner and Competitors

In consideration of the submission of drawings in this competition (here insert the name of the owner or of the body duly authorized to enter into contract on behalf of the owner), hereinafter called the Owner, agrees with the competitors jointly and severally that the owner will, within .... days of the date set for the submission of drawings, make an award of the commission to design and supervise the work forming the subject of this competition to one of those competitors who submit drawings in consonance with the mandatory requirements of this programme, and will thereupon pay him, on account of his services as architect, one-tenth of his total estimated fee as stated below. And further in consideration of the submission of drawings as aforesaid and the mutual promises enumerated in the subjoined Conditions of Contract between Architect and Owner, the owner agrees and each competitor agrees, if the award be made in his favor, immediately to enter into a contract containing the Conditions here following, and until such contract is executed to be bound by the said Conditions.

Conditions of Contract between Architect and Owner

Article I. Duties of the Architect

(1) **Design.** The architect is to design the entire building and its immediate surroundings and is to design or direct the design of its constructive, engineering and decorative work and its fixed equipment and, if further retained, its movable furniture and the treatment of the remainder of its grounds.

(2) **Drawings and Specifications.** The architect is to make such revision of his competitive scheme as may be necessary to complete the preliminary studies; and he is to provide drawings and specifications necessary for the conduct of the work. All such instruments of service are and remain the property of the architect.

(3) **Administration.** The architect is to prepare or advise as to all forms connected with the making of proposals and contracts, to issue all certificates

payment, to keep proper accounts and generally to discharge the necessary lministrative duties connected with the work.

(4) **Supervision.** The architect is to supervise the execution of all the work mmitted to his control.

ARTICLE II. DUTIES OF THE OWNER

(1) **Payments.** The owner is to pay the architect for his services a sum jual to .... per cent * upon the cost of the work. (The times and amounts payments should be here stated.) †

(2) **Reimbursements.** The owner is to reimburse the architect, from time time, the amount of expenses necessarily incurred by him or his deputies ile traveling in the discharge of duties connected with the work.

(3) **Service of Engineers.** The owner is to reimburse the architect the cost the services of such engineers for heating, mechanical and electrical work as e specifically provided for in each programme. The selection of such engineers d their compensation shall be subject to the approval of the owner.

(4) **Information, Clerk of the Works, etc.** The owner is to give all inrmation as to his requirements; to pay for all necessary surveys, borings and ts, and for the continuous services of a clerk of the works, whose competence approved by the architect.

Standard Form of Competition-Programme ‡

The following standard form of COMPETITION-PROGRAMME, prepared by The nerican Institute of Architects, contains those provisions which the Institute nsiders essential to the fair and equitable conduct of a competition. The stitute in no way assumes or attempts to dictate an OWNER's course in conicting a competition; it claims only the right to control its own members, and ving found by experience the danger to the interests of both OWNER and MPETITOR from a competition in which such provisions are lacking, it perits no member to take part in any competition which does not meet those ential conditions, and the programme of which has not been specifically proved. A competition should be of such form as to establish equitable relans between the OWNER and the COMPETITOR. To insure this, the requirents should be clear and definite; the competency of the COMPETITORS should assured; the agreement between the OWNER and COMPETITORS should be finite, as becomes a plain statement of business relations; and the judgment ould be based on expert knowledge. The following programme will, if adhered be duly approved by the Institute SUBCOMMITTEES ON COMPETITIONS for e various chapters of the Institute, and by the STANDING COMMITTEE ON MPETITIONS of the Institute.

* The percentage inserted should be in accord with good practice.

† Good practice has established the payments on account as follows: Upon completion the preliminary studies one-fifth of the total estimated fee less the previous payment; n completion of contract-drawings and specifications two-fifths additional of such fee; other drawings, for supervision and for administration, the remainder of the fee, n time to time, in proportion to the progress of the work.

‡ As authorized by the 48th annual convention, 1914. The American Institute of hitects, Document, Series A, No. 115, Washington, D. C., January, 1918. Reprinted permission.

Programme of Competition for

....

(Insert name of proposed building)

NOTE. Throughout this programme the word OWNER is used to indicate either the owner in person, or those to whom he has delegated his powers.

PART I

(1) **Proposed Building.** The..

(Insert name of owner)

proposes to erect a new..

(Insert name of building)

on the site at..

(Insert location)

(2) **Authority.** The..

(Insert name of owner)

has (delegated to..

(Insert name or names of individuals)

authority to select an ARCHITECT to prepare the plans for, and supervise the erection of the building.

NOTE. If authority for the erection of the proposed building is granted by act of legislature, ordinance, etc., it is desirable to make clear the source of such authority.

(3) **Architectural Adviser.** The OWNER has appointed as his expert PROFESSIONAL ADVISER..

(Insert name and address of adviser)

to prepare this programme and to act as his ADVISER in the conduct of this competition.

NOTE. No competition shall be instituted without the aid of a competent adviser. He should be an architect of the highest standing and his selection should be the Owner's first step. He should be chosen with the greatest care, as the success of the competition will depend largely upon his experience and ability. The duties of the expert are to advise those who hold the competition in regard to its form and terms, to draw up the programme, to advise in choosing the COMPETITORS, to answer inquiries from COMPETITORS and in general to direct the competition.

(4) **Competitors.** Participation in this competition is limited (A), to the following ARCHITECTS:..

(Insert names of invited competitors)

or
and (B) To such ARCHITECTS as shall have made application on or before.....

....

(Insert date)

accompanied by evidence of their education and experience, satisfactory to the OWNER and the PROFESSIONAL ADVISER. It is agreed that the names of all those admitted to the competition shall be made public on or before..........

....

(Insert date)

The OWNER agrees that he will admit no one as a COMPETITOR to whom he is not willing to award the commission to erect the building, in case of his success in the competition.

(5) **Jury of Award.** The Owner agrees that there will be a Jury of Award
l) which will consist of the following members:.........................
....
(Insert names of jury)

r (B) which will consist of..............members. Of these, the Owner
(Insert number)

s appointed the following:...
....and
(Insert names of those so selected)

e Competitors will select the remaining members of the Jury.

NOTE. To insure a just and wise award and to protect the interests of both the wner and the Competitors, the drawings should be submitted to a Jury chosen to cure expert knowledge and freedom from personal bias. The Jury shall consist of at ist three members, one of whom must, and the majority of whom should, be practicing chitects, for example, a layman and an architect selected by the Owner or the Building mmittee, and an architect selected by the Competitors. For work of great imporace it is desirable to increase the size of the Jury, adding to it architects and specially alified laymen. Some of the advantages of a Jury so constituted are that it thoroughly iderstands and can explain the intent of the drawings, and discovers from them their thor's skill in design, arrangement and construction. Because of its expert knowledge, judgment on the merits of the designs submitted is of the highest value to the Owner. e adoption of the recommendation that the architectural members of the Jury be in e majority, is not necessarily a cause of expense, for the reason that in order to insure e proper conduct of competitions, many architects of standing are willing, if the occasion irrants, to serve as Jurors without payment, other than actual expenses. It is cusmary and desirable that the Competitors should elect one or more of the architectural embers of the Jury. It is not advisable that the Professional Adviser, who has awn up the programme, be permitted to vote as a member of the Jury, although he ay with advantage take part in the deliberations of the Jury.

(6) **Authority of Jury.** The Owner agrees that the Jury above named, or lected as above provided, will have authority to make the award and that its cision in the matter shall be final. Moreover, this Jury will make an award one of those taking part in this competition, unless no design is submitted iich fulfils the mandatory requirements of this programme. The Owner rther agrees to employ as architect for the work as more fully set forth hereiner, the author of the design selected by the Jury as its first choice.

NOTE. If, under the law, authority to make the award cannot be delegated to the ry, the following form should be substituted for Section (6):

The Owner agrees that the Jury above named or selected as above provided, will ect the design which appears to it to be the most meritorious and make a written report the Owner, designating it by number. The Owner will then consider this design the report of the Jury and will thereupon, without learning the identity of the Comitors, select as the winner of the competition the author of the design selected by Jury, unless in his judgment there be cause to depart from such selection, in which e he will, still without learning the identity of the Competitors, select one of the other igns submitted in competition. The Owner further agrees that he will pay to the thor of the design designated as most meritorious by the Jury, in case he should not appointed Architect of the building, a prize of $.................................
(State amount of prize)

The opening of the envelope containing the name of the author of the design selected the Owner will automatically close the contract between him and the Owner, printed Part III hereof.

(7) **Examination of Designs and Award.** The Professional Adviser will amine the designs to ascertain whether they comply with the mandatory re-

quirements of the programme, and will report to the JURY any instance of failure to comply with these mandatory requirements. The OWNER further agrees that the JURY will satisfy itself of the accuracy of the report of the PROFESSIONAL ADVISER, and will place out of competition and make no award to any design which does not comply with these mandatory requirements. The JURY will carefully study the programme and any modifications thereof, which may have been made through communications (see Section (12)), and will then consider the remaining designs, holding at least two sessions on separate days, and considering at each session all the drawings in competition, and will make the award and the classification of prize-winners, if prizes are given, by secret ballot, and by majority vote, before opening the envelopes which contain the names of the COMPETITORS. In making the award the JURY will thereby affirm that it has made no effort to learn the identity of the various COMPETITORS, and that it has remained in ignorance of such identity until after the award was made. The opening of the envelope containing the name of the author of the selected design will automatically close the contract between him and the OWNER, printed as Part III hereof.

(8) **Report of the Jury.** The JURY will make a full report which will state its reasons for the selection of the winning design and its reason for the classification of the designs placed next in order of merit, and a copy of this report, accompanied by the names of prize-winners, if prizes are given, will be sent by the PROFESSIONAL ADVISER to each COMPETITOR. Immediately upon the opening of the envelopes, the PROFESSIONAL ADVISER will notify all COMPETITORS, by wire, of the result of the competition.

(9) **Compensation to Competitors.** The OWNER agrees to pay to the successful COMPETITOR within ten days of the judgment, on account of his fee for services as ARCHITECT, one-tenth of his total estimated fee.

In full discharge of his obligation to them (in case prizes or fees are offered), the OWNER agrees:

(A) To pay the following prizes to those ranked by the JURY next to the successful design: To the design placed second $........, to the design placed third $........, to the design placed fourth $........, to the design placed fifth $........., etc., within ten days of the judgment, or

(B) To pay to each of the COMPETITORS invited to take part in this competition, other than the successful COMPETITOR, a fee of $........ within ten days of the judgment.

(10) **Exhibition of Drawings.** It is agreed that no drawings shall be exhibited or made public until after the award of the JURY. There will be a public exhibition of all drawings after the judgment, and all drawings, except that of the successful COMPETITOR, will be returned to their authors at the close thereof.

(11) **Use of Features of Unsuccessful Designs.** Nothing original in the unsuccessful designs shall be used without consent of, or compensation to, the author of the design in which it appears. In case the OWNER desires to make use of any individual feature of an unsuccessful design, the same may be obtained by adequate compensation to the designer, the amount of such compensation to be determined in consultation with the author and the PROFESSIONAL ADVISER.

(12) **Communications. (Mandatory.)** If any COMPETITOR desires information of any kind whatever in regard to the competitition, or the programme, he shall ask for this information by anonymous letter addressed to the PROFESSIONAL ADVISER, and in no other way, and a copy of this letter and the

answer thereto will be sent simultaneously to each COMPETITOR, but no request received after ....

(Insert date)

will be answered.

(13) **Anonymity of Drawings. (Mandatory.)** The drawings to be submitted shall bear no name or mark which could serve as a means of identification, nor shall any such name or mark appear upon the wrapper of the drawings, nor shall any COMPETITOR directly or indirectly reveal the identity of his designs, or hold communication regarding the competition with the OWNER or with any member of the BUILDING COMMITTEE or of the JURY, or with the PROFESSIONAL ADVISER, except as provided for under COMMUNICATIONS. It is understood that in submitting a design, each COMPETITOR thereby affirms that he has complied with the foregoing provisions in regard to anonymity and agrees that any violation of them renders null and void this agreement and any agreement arising from it. With each set of drawings must be enclosed a plain, opaque, sealed envelope without any superscription or mark of any kind, same containing the name and address of the COMPETITOR. These envelopes shall be opened by the PROFESSIONAL ADVISER after the final selection has been made, and preferably in the presence of the JURY.

(14) **Delivery of Drawings. (Mandatory.)** The drawings submitted in this competition shall be securely wrapped, addressed to the PROFESSIONAL ADVISER at ....
.... in plain lettering and

(Insert address for delivery of drawings)

with no other lettering thereon, and delivered at this address not later than
....

(Insert date and hour)

In case drawings are sent by express, they may be delivered to an express company at the above date and hour, in which case the express company's receipt, bearing date and hour, shall be mailed immediately to the PROFESSIONAL ADVISER as evidence of delivery.

PART II

(15) **Site.** The site of the building is as follows....................

(Insert description of site, and provide topographical map giving dimensions, grades, etc.)

NOTE. The site should be carefully described and a survey of the property should be attached and included as part of the programme. Conditions pertaining to the site and to neighboring buildings frequently become determining factors in a design. Photographs showing surrounding buildings and landscape-conditions may with advantage be included.

(16) **Cost. (Mandatory.)** For the purpose of this competition the cost of the building shall be figured at.........cts per cu ft, and the total thereof

(Insert number)

figured on this basis shall not exceed ....

(Insert limit of cost)

(17) **Cubage. (Mandatory.)** Cubage shall be so computed as to show as exactly as possible the actual volume of the building, calculated from the finished level or levels of the lowest floor to the highest points of the roofs, and contained within the outside surfaces of the walls. Pilasters, cornices, balconies and other similar projections shall not be included. Porticos with engaged columns and similar projections shall be taken as solids and figured to the outer face of the columns. When columns are free-standing, one-half of the volume of the porti-

cos shall be taken. There shall also be included in the cubage the actual volume of all parapets, towers, lanterns, dormers, vaults, and other features adding to the bulk of the building, also the actual volume of exterior steps above grade. Light-wells of an area of less than 400 sq ft shall not be deducted. In calculating cubage, account shall be taken of variations in the exterior wall-surface, as for example, the projection of a basement-story beyond the general line of the building. A figured diagram showing method adopted in cubing shall accompany each set of drawings.

(18) **Drawings.** **(Mandatory.)** The drawings submitted shall be made according to the following list, at the scale given, and rendered as noted; and no other drawings than these shall be submitted:

....

(Insert list, scale and method of rendering)

NOTE. The drawings submitted should be the least number necessary to set forth clearly the solution of the problem, and the scale of these drawings the smallest compatible with the requirement that the intention of each COMPETITOR be made clear to an expert JURY. Where the number and scale of drawings is reduced to the minimum, and simple methods of rendering imposed, the COMPETITORS are enabled to devote their time and energy to the study of the problem, which is the serious business of a competition, instead of upon draughtsmanship and rendering, which when carried beyond a certain point, are of no value whatever in determining the fitness of the COMPETITORS to handle the work of erecting the building, for which the competition is being held.

PART III

Agreement between Owner and Competitors

In consideration of the submission of drawings in this competition, and the mutual promises enumerated in the subjoined CONDITIONS OF CONTRACT BETWEEN ARCHITECT AND OWNER the OWNER agrees, and each COMPETITOR agrees if the award be made in his favor, immediately to enter into a contract containing all the CONDITIONS here following, and until such contract is executed, to be bound by the said CONDITIONS.

Conditions of Contract between Architect and Owner

Duties of the Architect

(1) **Design.** The ARCHITECT is to design the entire building and its immediate surroundings and is to design or direct the design of its constructive, engineering and decorative work and its fixed equipment and, if further retained, its movable furniture and the treatment of the remainder of its grounds.

(2) **Drawings and Specifications.** The ARCHITECT is to make such revision of his competitive scheme as may be necessary to complete the preliminary studies; and he is to provide drawings and specifications necessary for the conduct of the work. All such instruments of service are and remain the property of the ARCHITECT.

(3) **Administration.** The ARCHITECT is to prepare or advise as to all forms connected with the making of proposals and contracts, to issue all certificates of payment, to keep proper accounts and generally to discharge the necessary administrative duties connected with the work.

(4) **Supervision.** The ARCHITECT is to supervise the execution of all the work committed to his control.

Duties of the Owner

(5) **Payments.** The OWNER is to pay the ARCHITECT for his services a um equal to........per cent upon the cost of the work.

NOTE. The percentage inserted should be in accord with good practice. The times d amounts of payments should be here stated. Good practice has established the yments on account as follows: Upon completion of the preliminary studies one-fifth the total estimated fee less the previous payment; upon completion of contract-drawings d specifications two-fifths additional of such fee; for other drawings, for supervision d for administration, the remainder of the fee, from time to time, as the work progresses.

(6) **Reimbursements.** The OWNER is to reimburse the architect from time time, the amount of expenses necessarily incurred by him or his deputies hile traveling in the discharge of duties connected with the work.

(7) **Service of Engineers.** The OWNER is to reimburse the ARCHITECT, e cost of the services of ENGINEERS for...
nsert nature of work for which the OWNER agrees that ENGINEERS shall be employed at s expense)

The selection of such ENGINEERS and their compensation shall be subject to e approval of the OWNER.

(8) **Information, Clerk of the Works, Etc.** The OWNER is to give all in- rmation as to his requirements; to pay for all necessary surveys, borings and sts, and for the continuous services of a clerk of the works whose competence approved by the ARCHITECT.

PART IV

Requirements of the Building

NOTE. For the same reason that elaborate drawings are undesirable, it is advisable avoid lengthy and detailed instructions as to the desired accommodations, as they nfuse the problem and hamper the COMPETITORS; and the OWNER loses thereby the nefit he might gain in allowing the COMPETITORS freedom to develop solutions which ey would not otherwise be at liberty to suggest. It should be borne in mind that either e cost of the building, as determined by its cubical contents, should be fixed, or the quirements of the OWNER in regard to the design, materials of construction, dimensions rooms, etc., should be fixed, but not both. If, on the one hand, the cubical contents d cost is fixed, it should be stated that the requirements of the OWNER must be adhered as closely as possible by COMPETITORS; if, on the other hand, the requirements of the WNER are definitely fixed, it may be stated that the cubical contents of each design, hile not limited, will be taken into consideration in making the award. In case the es of certain rooms, etc., are definitely fixed, the word MANDATORY should be placed the head of the paragraph referring to these rooms.

Here should follow a list of rooms required, together with sizes and other data hich apply to the building under consideration.

....
....
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....

THE STANDARD DOCUMENTS OF THE AMERICAN INSTITUTE OF ARCHITECTS*

Introductory Notes. This introductory paragraph is from an article† by Clipston Sturgis, President of The American Institute of Architects. "For many years builders and owners have commonly used an agreement recognized as inadequate and imperfect, and one apt to lead to serious misunderstandings, if not to legal difficulties. Architects entrusted with important work and its accompanying responsibilities have endeavored to have agreements drawn which would adequately safeguard the interests involved. When, some nine years ago (1906), the Institute attempted to prepare a new standard agreement, it found already in use a considerable number of forms prepared by architects, differing in detail but agreeing in one main point. This one point was that the contract and the conditions of the contract should be treated as two branches of the same agreement, not as one document, nor yet as two. The contract was to be as brief as possible, stating simply what the obligation was. The conditions of the contract, complicated and involved, yet essential to the contract, were of necessity comparatively lengthy. The most difficult part of the work, surveying the field and breaking out the way, was done by the Committees on Contracts and Specifications during the years 1906 to 1911, and resulted in the first edition of the STANDARD DOCUMENTS, published in 1911. At that time some thought the problem solved; others thought it but an important step forward; which latter proved to be the fact. These first documents, excellent as they were as text-books, were not suitable for everyday use. The Institute again took up the problem, this time with the definite aim to produce a document which should entirely replace the uniform agreement when the contract for its publication expired in May 1915. This has been done and the carefully studied AGREEMENT and CONDITIONS OF THE CONTRACT presented to the convention in December, 1914, have been further studied and improved and are now (1915) on the market for general use. In the final study between January and May, 1915, the Institute had the advantages of coöperation with representatives of many of the building trades and the advice of counsel representing the Institute and counsel representing the building trades. The document, like its predecessor, will now come to the test of actual use. It will prove to be imperfect and revised sections will be necessary, but it is believed to be in the main a fair and comprehensive agreement and one that is practical and fit for general use. Architects everywhere are urged to use and test this form, and criticism from owners and builders will be gladly received and considered. In addition to this most important document the committee has prepared and the Institute has published a form of BOND, a LETTER OF ACCEPTANCE by a contractor of a sub-contractor's bid, and an AGREEMENT between a contractor and sub-contractor. Many architects who have done work on which a bond has been required have been surprised at the ease with which the obligations of the bond could be evaded. In most cases, because someone, architect, contractor, or owner, had invalidated the bond. The new form of bond is prepared for insuring, as far as possible, that the bonding company shall discharge its obligations and protect the owner who pays for this protection. The LETTER FROM CONTRACTOR TO SUB-CONTRACTOR is intended to provide a simple form whereby the mutual obligations of the two shall be clearly defined. The AGREEMENT BETWEEN CONTRACTOR AND SUB-CONTRACTOR accomplishes the same purpose in a somewhat more formal way."

* Third Edition, copyrighted by The American Institute of Architects, 1915–1918, and inserted here by permission.

† Published in the Journal of The American Institute of Architects, June, 1915.

The Development of the Standard Documents. In the year 1887 The merican Institute of Architects, the Western Association of Architects and the ational Association of Builders, thinking it desirable to establish better practice the matter of building contracts, undertook the preparation of a form of conact satisfactory to all. Under the name of THE UNIFORM CONTRACT this form tained wide acceptance and has been long in use. About the year 1907, feeling at practice had advanced to a point no longer fully reflected by the UNIFORM ONTRACT, the Institute undertook a general study of the subject with a view developing a form of contract clear in thought, equitable, applicable to work almost all classes, binding in law and a standard of good practice. The work as entrusted to the Standing Committee on Contracts and Specifications, who ent four years on it, studying the UNIFORM CONTRACT and forms in use by some irty well-known architects, and submitted various drafts for criticism to the apters of the Institute and to engineers, contractors and architects throughout e country. The documents were prepared under the advice of Francis Fisher ane, counsel for the Institute, and Ernest Eidlitz, and with the able and careful iticism of Professor Samuel Williston of the Harvard Law School, and with the sistance of James W. Pryor, in their editing. The Institute gave its approval the work in 1911. The Standing Committee on Contracts and Specifications, uring the preparation of the first edition of the STANDARD FORMS, consisted of rosvenor Atterbury, Chairman; Allen B. Pond, Secretary; Frank Miles Day, illiam A. Boring, Frank C. Baldwin, Frank W. Ferguson, Alfred Stone and . L. Heins. Criticisms of the first edition of the DOCUMENTS were invited by e Institute and during the year 1913 a group of architects and builders in Boson, known as the Joint Committee of the Boston Society of Architects, and of e Master Builders' Association, gave much sincere study to the subject. At e same time the National Association of Builders' Exchange offered a detailed riticism of the documents.

In 1914 the Institute instructed its Standing Committee on Contracts and pecifications to undertake a general revision with a view to making the CONITIONS simpler in wording and more equitable. The committee was empowered hold conferences with organizations so desiring. Subcommittees for the terriory of the several chapters of the Institute were appointed and collaborated with he standing committee. The Boston group presented its ideas in the form of an ntirely new draft which proved of high value and its Chairman, W. Stanley 'arker, was present with the Standing Committee at nearly all its meetings. he Committee had a joint meeting with representatives of the National Assoiation of Builders' Exchanges and thereafter the counsel of the Association, V. B. King, and the counsel of the Institute, Louis Barcroft Runk, collaboated most effectively with the committee. The GENERAL CONDITIONS were ntirely rewritten and in response to the strong desire of contractors and subconractors, the principle of GENERAL ARBITRATION, subject to limitations in the locuments, was adopted, and provisions relative to the RELATIONS OF THE CONRACTOR AND HIS SUBCONTRACTORS were included in the documents. After much tudy, conference and criticism, a draft of the second edition was issued by uthority of the Institute, April 1, 1915. During the revision of the documents, he Standing Committee on Contracts and Specifications consisted of Frank Miles Day, Chairman; Allen B. Pond, Sullivan W. Jones, Clarence A. Martin, Norman M. Isham, Octavius Morgan, Thomas Nolan, A. O. Elzner, M. B. Medary, Jr., Jos. Evans Sperry, Frank W. Ferguson and Samuel Stone.

The Construction of the Standard Documents. An AGREEMENT, and DRAWINGS and SPECIFICATIONS are the necessary parts of a building contract. Many conditions of a general character may be placed at will in the AGREEMENT

or in the SPECIFICATIONS. It is, however, wise to assemble them in a single document and, since they have as much bearing on the DRAWINGS as on the SPECIFICATIONS, and even more on the business relations of the contracting parties, they are properly called the GENERAL CONDITIONS OF THE CONTRACT. As the AGREEMENT, GENERAL CONDITIONS, DRAWINGS and SPECIFICATIONS are the constituent elements of the CONTRACT and are acknowledged as such in the AGREEMENT, they are correctly termed the CONTRACT DOCUMENTS. Statements made in any one of them are just as binding as if made in the AGREEMENT. The Institute's forms, although intended for use in actual practice, should also be regarded as a code of reference representing the judgment of the Institute as to what constitutes good practice and as such they may be drawn upon by architects in improving their own forms. Although the forms are suited for use in connection with a single or general contract, they are equally applicable to an operation conducted under separate contracts.

Titles of the Standard Contract Documents.* The new STANDARD CONTRACT DOCUMENTS of The American Institute of Architects are now on sale† by dealers in office and drafting-supplies in all the large cities of the country, and replace the old UNIFORM CONTRACT. The following are the titles of the STANDARD DOCUMENTS: A. 1. FORM OF AGREEMENT AND A. 2. GENERAL CONDITIONS OF THE CONTRACT. B. BOND OF SURETYSHIP. C. FORM OF SUBCONTRACT. D. LETTER OF ACCEPTANCE OF SUBCONTRACTOR'S PROPOSAL. A cover in heavy paper with valuable EXPLANATORY NOTES is sent without charge with each complete set of the documents. These documents have received the full approval of the Institute, through its conventions, board of directors and officers. They are the outcome of nine years of continuous work by the Standing Committee on Contracts and Specifications. This committee, comprising some of the ablest American architects, was assisted by the Institute's thirty-nine chapters; advised by eminent legal specialists in contract law and aided by representatives of the Building and Trade Associations of the United States. The Standard Documents have received the formal approval of the

* Third Edition, copyrighted by the American Institute of Architects.

† **Notice to Architects, Builders and Contractors.** The CONTRACT FORMS may be obtained singly or in lots from the local dealers. If your dealer cannot supply you send your order and his name to The Executive Secretary, A. I. A., The Octagon, Washington, D. C. All orders must include the necessary remittance irrespective of A. I. A. membership and irrespective of commercial standing of purchaser. The Institute has adopted these CASH TERMS, from which no exception will be made to anybody, in order to reduce cost of accountancy and thereby reduce expense to the user. Remittances may be by check, money-order, cash, or stamps.

Prices for Single Copies: Agreement and General Conditions in cover, \$0.14; General Conditions without Agreement, \$0.10; Agreement without General Conditions, \$0.0[illegible]; Bond of Suretyship, \$0.02; Form of Subcontract, \$0.03; Letter of Acceptance of Subcontractor's Proposal, \$0.02; Cover (heavy paper, with valuable notes), \$0.01; Complete set in cover, \$0.20. A Trial set will be delivered upon receipt of ten 2-cent stamps.

Prices for Quantities and Discounts to Architects, Builders and Contractors. Orders for quantities are subject to the following discounts (which are also given by all dealers):

Five per cent on lots of 100 (one kind or assorted); 10% on lots of 500 (one kind or assorted); 15% on lots of 1 000 (one kind or assorted). As these DOCUMENTS are printed on sheets, 8½ by 11 ins, and in large quantities, they cannot be supplied with any individual names or printing different from the standard forms. The Institute does not wish to encourage the use of the AGREEMENT with general conditions other than those endorsed by it, but on request will sell the AGREEMENTS separate from the STANDARD GENERAL CONDITIONS at 3 cts each.

ional Association of Builders' Exchanges, the National Association of Master mbers, the National Association of Sheet Metal Contractors of the United es, the National Electrical Contractors' Association of the United States, National Association of Marble Dealers, the Building Granite Quarries ciation, the Building Trades Employers' Association of the City of New k, and the Heating and Piping Contractors' National Association.

1. THE STANDARD FORM OF AGREEMENT BETWEEN CONTRACTOR AND OWNER*

SSUED BY THE AMERICAN INSTITUTE OF ARCHITECTS †

is form has been approved by the National Association of Builders' Exchanges, The onal Association of Master Plumbers, the National Association of Sheet Metal tractors of the United States, the National Electrical Contractors' Association of United States, the National Association of Marble Dealers, the Building Granite rries Association, the Building Trades Employers' Association of the City of New ., and the Heating and Piping Contractors' National Association.

S AGREEMENT, made the.. of...............in the year Nineteen Hundred and............... nd between....................(Two blank lines) ‡ .... inafter called the Contractor, and..............(Two blank lines)........ ....hereinafter called the Owner NESSETH, that the Contractor and the Owner for the considerations inafter named agree as follows:

rticle 1. The Contractor agrees to provide all the materials and to perform e work shown on the Drawings and described in the Specifications entitled

ere insert the caption descriptive of the work as used in the Proposal, General Cons, Specifications, and upon the Drawings.)

........................(Five blank lines)............................

ared by...............(Two blank lines).......................... g as, and in these Contract Documents entitled the Architect, and to do ything required by the General Conditions of the Contract, the Specifica- and the Drawings.

ticle 2. The Contractor agrees that the work under this Contract shall bstantially completed.

(Here insert the date or dates of completion, and stipulations as to liquidated damages if any.)

........................(Eight blank lines)...........................

ticle 3. The Owner agrees to pay the Contractor in current funds for the rmance of the Contract.

....
..................................($..................) subject ditions and deductions as provided in the General Conditions of the Con-

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or use when a stipulated sum forms the basis of payment.
otted lines, as indicated, are in the standard documents and are omitted here to space.

tract and to make payments on account thereof as provided therein, as fol
On or about the ....day of each month..................
per cent of the value, proportionate to the amount of the Contract, of la
and materials incorporated in the work.................................
...........................up to the first day of that month as estim
by the Architect, less the aggregate of previous payments. On substan
completion of the entire work, a sum sufficient to increase the total paym
to...........................per cent of the contract price, and..........
................days thereafter, provided the work be fully completed and t
Contract fully performed, the balance due under the Contract.

...........................(Five blank lines)...........................

Article 4. The Contractor and the Owner agree that the General Conditi of the Contract, the Specifications and the Drawings, together with this Agr ment, form the Contract, and that they are as fully a part of the Contract a hereto attached or herein repeated; and that the following is an exact enume tion of the Specifications and Drawings:

........................(Thirty-five blank lines)........................

The Contractor and the Owner for themselves, their successors, executo administrators and assigns, hereby agree to the full performance of the coven herein contained.

IN WITNESS WHEREOF they have executed this agreement, the day a year first above written.

....
....
....
....
....
....
....

A. 2. THE GENERAL CONDITIONS OF THE CONTRACT*

STANDARD FORM OF THE AMERICAN INSTITUTE OF ARCHITECTS

This form has been approved by the National Association of Builders' Exchanges, National Association of Master Plumbers, the National Association of Sheet M Contractors of the United States, the National Electrical Contractors' Association of t United States, the National Association of Marble Dealers, the Building Granite Qua Association, the Building Trades Employers' Association of the City of New Yo and the Heating and Piping Contractors' National Association.

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\rt. 1. Principles and Definitions.

a) The Contract Documents consist of the Agreement, the General Con- ons of the Contract, the Drawings and Specifications, including all modi- tions thereof incorporated in the documents before their execution. These n the Contract.

b) The Owner, the Contractor and the Architect are those named as such in Agreement. They are treated throughout the Contract Documents as if h were of the singular number and masculine gender.

c) The term Subcontractor, as employed herein, includes only those having rect contract with the Contractor and it includes one who furnishes material ked to a special design according to the plans or specifications of this work, does not include one who merely furnishes material not so worked.

d) Written notice shall be deemed to have been duly served if delivered in son to the individual or to a member of the firm or to an officer of the corpora- for whom it is intended, or if delivered at or mailed to the last business ress known to him who gives the notice.

e) The term "work" of the Contractor or Subcontractor includes labor or terials or both.

f) All time-limits stated in the Contract Documents are of the essence of contract.

g) The law of the place of building shall govern the construction of this tract.

\rt. 2. Execution, Correlation and Intent of Documents. The Con- t Documents shall be signed in duplicate by the Owner and Contractor. In of failure to sign the General Conditions, Drawings or Specifications the hitect shall identify them. The Contract Documents are complementary, what is called for by any one shall be as binding as if called for by all. intention of the documents is to include all labor and materials reasonably essary for the proper execution of the work. It is not intended, vever, that materials or work not covered by or properly inferable from any ding, branch, class or trade of the specifications shall be supplied unless inctly so noted on the drawings. Materials or work described in words ch so applied have a well-known technical or trade meaning shall be held efer to such recognized standards.

\rt. 3. Detail Drawings and Instructions. The Architect shall furnish, h reasonable promptness, additional instructions, by means of drawings or erwise, necessary for the proper execution of the work. All such drawings

and instructions shall be consistent with the Contract Documents, true devel-ments thereof, and reasonably inferable therefrom. The work shall be exec in conformity therewith and the Contractor shall do no work without pr drawings and instructions. In giving such additional instructions, the Archi shall have authority to make minor changes in the work, not involving ex costs, and not inconsistent with the purposes of the building. The Contract and the Architect, if either so requests, shall jointly prepare a schedule, subj to change from time to time in accordance with the progress of the wor fixing the latest dates at which the various detail drawings will be requir and the Architect shall furnish them in accordance with that schedule. Und like conditions, a schedule shall be prepared, fixing dates for the submissi of shop drawings, for the beginning of manufacture and installation of materi and for the completion of the various parts of the work.

Art. 4. Copies Furnished. Unless otherwise provided in the Contrac Documents the Architect will furnish to the Contractor, free of charge, a copies of drawings and specifications reasonably necessary for the executio the work.

Art. 5. Shop Drawings. The Contractor shall submit, with such promp ness as to cause no delay in his own work or in that of any other contracto two copies of all shop or setting drawings and schedules required for the wor of the various trades and the Architect shall pass upon them with reasonab promptness. The Contractor shall make any corrections required by th Architect, file with him two corrected copies and furnish such copies as ma be needed. The Architect's approval of such drawings or schedules shall n relieve the Contractor from responsibility for deviations from drawings specifications, unless he has in writing called the Architect's attention to su deviations at the time of submission, nor shall it relieve him from responsibili for errors of any sort in shop drawings or schedules.

Art. 6. Drawings and Specifications on the Work. The Contractor sha keep one copy of all drawings and specifications on the work, in good ord available to the Architect and to his representatives.

Art. 7. Ownership of Drawings and Models. All drawings, specificati and copies thereof furnished by the Architect are his property. They are n to be used on other work and, with the exception of the signed contract s are to be returned to him on request, at the completion of the work. A models are the property of the Owner.

Art. 8. Samples. The Contractor shall furnish for approval all samples directed. The work shall be in strict accordance with approved samples.

Art. 9. The Architect's Status. The Architect shall have general sup vision and direction of the work. He is the agent of the Owner, only to the e tent provided in the Contract Documents and when in special instances he authorized by the Owner so to act, and in such instances he shall, upon reque show the Contractor written authority. He has authority to stop the w whenever such stoppage may be necessary to insure the proper execution of Contract. As the Architect is, in the first instance, the interpreter of Contract and the judge of its performance, he shall side neither with the Ow nor with the Contractor, but shall use his powers under the Contract to enf its faithful performance by both. In case of the termination of the empl ment of the Architect, the Owner shall appoint a capable and reputable Archi whose status under the contract shall be that of the former Architect.

Art. 10. The Architect's Decisions. The Architect shall, within a reas able time, make decisions on all claims of the Owner or Contractor and on

r matters relating to the execution and progress of the work or the interpre-
n of the Contract Documents. The Architect's decisions, in matters relating
tistic effect, shall be final, if within the terms of the Contract Documents.
pt as above or as otherwise expressly provided in these General Conditions
the specifications, all the Architect's decisions are subject to arbitration.

t. 11. **Foreman, Supervision.** The Contractor shall keep on the work a
etent foreman and any necessary assistants, all satisfactory to the
itect. The foreman shall not be changed except with the consent of the
itect, unless the foreman proves to be unsatisfactory to the Contractor
ceases to be in his employ. The foreman shall represent the Contractor
s absence and all directions given to him shall be as binding as if given to
Contractor. Important directions shall be confirmed in writing to the Con-
or. Other directions shall be so confirmed on written request in each
The Contractor shall give efficient supervision to the work, using his best
and attention. He shall carefully study and compare all drawings, specifi-
ns and other instructions and shall at once report to the Architect any error,
sistency, or omission which he may discover.

t. 12. **Materials, Appliances, Employees.** Unless otherwise stipulated,
Contractor shall provide and pay for all materials, labor, water, tools, equip-
, light and power necessary for the execution of the work. Unless other-
specified, all materials shall be new and both workmanship and materials
be of good quality. The Contractor shall, if required, furnish satisfactory
nce as to the kind and quality of materials. The Contractor shall not
oy on the work any unfit person or any one not skilled in the work assigned
m.

t. 13. **Inspection of Work.** The Owner, the Architect and their repre-
tives shall at all times have access to the work wherever it is in preparation
ogress and the Contractor shall provide proper facilities for such access and
spection. If the specifications, the Architect's instructions, laws, ordi-
s or any public authority require any work to be specially tested or
ved, the Contractor shall give the Architect timely notice of its readiness
spection, and if the inspection is by another authority than the Architect,
e date fixed for such inspection. Inspections by the Architect shall be
ptly made. If any such work should be covered up without approval or
nt of the Architect, it must, if required by the Architect, be uncovered
camination at the Contractor's expense. Reexamination of questioned
may be ordered by the Architect. If such work be found in accordance
the contract, the Owner shall pay the cost of reexamination and replace-
If such work be found not in accordance with the contract, through the
of the Contractor, the Contractor shall pay such cost, unless he shall show
the defect in the work was caused by another contractor, and in that
the Owner shall pay such cost.

t. 14. **Correction of Work Before Final Payment.** The Contractor
promptly remove from the premises all materials condemned by the
tect as failing to conform to the Contract, whether incorporated in the
or not, and the Contractor shall promptly replace and re-execute his own
in accordance with the Contract and without expense to the Owner and
bear the expense of making good all work of other contractors destroyed
maged by such removal or replacement. If the Contractor does not
re such condemned work and materials within a reasonable time, fixed by
en notice, the Owner may remove them and may store the material at
xpense of the Contractor. If the Contractor does not pay the expense

of such removal within five days thereafter, the Owner may, upon ten written notice, sell such materials at auction or at private sale and account for the net proceeds thereof, after deducting all the cost and exp that should have been borne by the Contractor.

Art. 15. Deductions for Uncorrected Work. If the Architect Owner deem it inexpedient to correct work injured or not done in accord with the Contract, the difference in value together with a fair allowance damage shall be deducted.

Art. 16. Correction of Work After Final Payment. Neither the final tificate nor payment nor any provision in the Contract Documents shall the Contractor of responsibility for negligence or faulty materials or work ship and he shall remedy any defects due thereto and pay for any damag other work resulting therefrom, which shall appear within a period of two from the time of installation. The Owner shall give notice of observed with reasonable promptness. All questions arising under this Article be decided under Articles 10 and 45.

Art. 17. Protection of Work and Property. The Contractor shall tinuously maintain adequate protection of all his work from damage and protect the Owner's property from injury arising in connection with this tract. He shall make good any such damage or injury, except such as directly due to errors in the Contract Documents. He shall adequately p adjacent property as provided by law and the Contract Documents.

Art. 18. Emergencies. In an emergency affecting the safety of life or structure or of adjoining property, not considered by the Contractor as the provisions of Article 17, then the Contractor, without special instructio authorization from the Architect or Owner, is hereby permitted to act, at discretion, to prevent such threatened loss or injury and he shall so act, wi appeal, if so instructed or authorized. Any compensation claimed to be d him therefor shall be determined under Articles 10 and 45 regardless of limitations in Article 25 and in the second paragraph of Article 24.

Art. 19. Contractor's Liability Insurance. The Contractor shall tain such insurance as will protect him from claims under workmen's comp tion acts and from any other claims for damages for personal injury, inc death, which may arise from operations under this contract, whether operations be by himself or by any subcontractor or anyone directly or ind employed by either of them. Certificates of such insurance shall be filed the Owner, if he so require, and shall be subject to his approval for ade of protection.

Art. 20. Owner's Liability Insurance. The Owner shall maintain insurance as will protect him from his contingent liability for damag personal injury, including death, which may arise from operations under Contract.

Art. 21. Fire Insurance. The Owner shall effect and maintain fire ance upon the entire structure on which the work of this contract is to be and upon all materials, in or adjacent thereto and intended for use the to at least eighty per cent of the insurable value thereof. The loss, if is to be made adjustable with and payable to the Owner as Trustee for it may concern. All policies shall be open to inspection by the Contr If the Owner fails to show them on request or if he fails to effect or mai insurance as above, the Contractor may insure his own interest and c the cost thereof to the Owner. If the Contractor is damaged by failure

ner to maintain such insurance, he may recover under Art. 39. If required writing by any party in interest, the Owner as Trustee shall, upon the urrence of loss, give bond for the proper performance of his duties. He shall osit any money received from insurance in an account separate from all his er funds and he shall distribute it in accordance with such agreement as the ties in interest may reach, or under an award of arbitrators appointed, one by Owner, another by joint action of the other parties in interest, all other cedure being in accordance with Art. 45. If after loss no special agreement is de, replacement of injured work shall be ordered under Art. 24. The Trustee ll have power to adjust and settle any loss with the insurers unless one the contractors interested shall object in writing within three working days the occurrence of loss and thereupon arbitrators shall be chosen as above. Trustee shall in that case make settlement with the insurers in accordance h the directions of such arbitrators, who shall also, if distribution by arbitra- is required, direct such distribution.

rt. 22. **Guaranty Bonds.** The Owner shall have the right to require Contractor to give bond covering the faithful performance of the contract the payment of all obligations arising thereunder, in such form as the Owner prescribe and with such sureties as he may approve. If such bond is ired by instructions given previous to the receipt of bids, the premium l be paid by the Contractor; if subsequent thereto, it shall be paid by the ner.

rt. 23. **Cash Allowances.** The Contractor shall include in the contract all allowances named in the Contract Documents and shall cause the work overed to be done by such contractors and for such sums as the Architect direct, the contract sum being adjusted in conformity therewith. The tractor declares that the contract sum includes such sums for expenses and it on account of cash allowances, as he deems proper. No demand for enses or profit other than those included in the contract sum shall be allowed. Contractor shall not be required to employ for any such work persons nst whom he has a reasonable objection.

rt. 24. **Changes in the Work.** The Owner, without invalidating the ract, may make changes by altering, adding to or deducting from the work, contract sum being adjusted accordingly. All such work shall be executed r the conditions of the original contract except that any claim for extension me caused thereby shall be adjusted at the time of ordering such change. xcept as provided in Articles 3, 9 and 18, no change shall be made unless in uance of a written order from the Owner signed or countersigned by the itect, or a written order from the Architect stating that the Owner has orized the change, and no claim for an addition to the contract sum shall alid unless so ordered.

he value of any such change shall be determined in one or more of the wing ways:

) By estimate and acceptance in a lump sum.
) By unit prices named in the contract or subsequently agreed upon.
) By cost and percentage or by cost and a fixed fee.
) If none of the above methods is agreed upon, the Contractor, provided eceive an order as above, shall proceed with the work, no appeal to arbitra- being allowed from such order to proceed.

cases (c) and (d), the Contractor shall keep and present in such form as Architect may direct, a correct account of the net cost of labor and mate- , together with vouchers. In any case, the Architect shall certify to the

amount, including a reasonable profit, due to the Contractor. Pending determination of value, payments on account of changes shall be made on Architect's certificate.

Art. 25. Claims for Extras. If the Contractor claims that any ins- tions, by drawings or otherwise, involve extra cost under this contract, he give the Architect written notice thereof before proceeding to execute the and, in any event, within two weeks of receiving such instructions, and procedure shall then be as provided in Art. 24. No such claim shall be unless so made.

Art. 26. Applications for Payments. The Contractor shall submit to Architect an application for each payment and, if required, receipts or vouchers showing his payments for materials and labor as required by Article If payments are made on valuation of work done, such application shall submitted at least ten days before each payment falls due, and, if requi the Contractor shall before the first application, submit to the Architect schedule of values of the various parts of the work, including quantities, ag gating the total sum of the contract, divided so as to facilitate payments subcontractors in accordance with Article 44 (e), made out in such form, if required, supported by evidence as to its correctness, as the Architect direct. This schedule when approved by the Architect, shall be used basis for certificates of payment, unless it be found to be in error. In appl for payments, the Contractor shall submit a statement based upon this sche and, if required, itemized in such form, and supported by such evidence, as Architect may direct, showing his right to the payment claimed.

Art. 27. Certificates and Payments. If the Contractor has made appli tion as above, the Architect shall, not later than the date when each paym falls due, issue to the Contractor a certificate for such amount as he dec to be properly due. No certificate issued nor payment made to the Contrac nor partial or entire use or occupancy of the work by the Owner shall be acceptance of any work or materials not in accordance with this contract. making and acceptance of the final payment shall constitute a waiver of claims by the Owner, otherwise than under Articles 16 and 29 of these ditions or under requirement of the specifications, and of all claims by Contractor, except those previously made and still unsettled. Should Owner fail to pay the sum named in any certificate of the Architect or in award by arbitration, upon demand when due, the Contractor shall receive addition to the sum named in the certificate, interest thereon at the legal in force at the place of building.

Art. 28. Payments Withheld. The Architect may withhold or, on acco of subsequently discovered evidence, nullify the whole or a part of any certif for payment to such extent as may be necessary to protect the Owner loss on account of:

(a) Defective work not remedied.

(b) Claims filed or reasonable evidence indicating probable filing of clai

(c) Failure of the Contractor to make payments properly to subcontra or for material or labor.

(d) A reasonable doubt that the contract can be completed for the bal then unpaid.

(e) Damage to another contractor under Article 40.

When all the above grounds are removed certificates shall at once be iss for amounts withheld because of them.

Art. 29. Liens. Neither the final payment nor any part of the reta

entage shall become due until the Contractor, if required, shall deliver to Owner a complete release of all liens arising out of this contract, or receipts ill in lieu thereof and, if required in either case, an affidavit that so far has knowledge or information the releases and receipts include all the r and material for which a lien could be filed; but the Contractor may, y subcontractor refuses to furnish a release or receipt in full, furnish a l satisfactory to the Owner, to indemnify him against any claim by lien or rwise. If any lien or claim remain unsatisfied after all payments are e, the Contractor shall refund to the Owner all moneys that the latter be compelled to pay in discharging such lien or claim, including all costs a reasonable attorney's fee.

t. 30. **Permits and Regulations.** The Contractor shall obtain and pay ll permits and licenses, but not permanent easements, and shall give all es, pay all fees, and comply with all laws, ordinances, rules and regulations ng on the work, as drawn and specified. If the Contractor observes that ings and specifications are at variance therewith, he shall promptly notify rchitect in writing, and any necessary changes shall be adjusted under 24. If the contractor performs any work knowing it to be con- to such laws, ordinances, rules and regulations, and without such notice to rchitect, he shall bear all costs arising thereform.

t. 31. **Royalties and Patents.** The Contractor shall pay all royalties license fees. He shall defend all suits or claims for infringement ny patent rights and shall save the Owner harmless from loss on nt thereof, except that the Owner shall be responsible for all such loss the product of a particular manufacturer or manufacturers is specified; f the Contractor has information that the article specified is an infringe- of a patent he shall be responsible for such loss unless he promptly gives information to the Architect or Owner.

t. 32. **Use of Premises.** The Contractor shall confine his apparatus, torage of materials and the operations of his workmen to limits indicated w, ordinances, permits, or directions of the Architect and shall not unreason- encumber the premises with his materials. The Contractor shall not load rmit any part of the structure to be loaded with a weight that will endanger fety. The Contractor shall enforce the Architect's instructions regarding advertisements, fires and smoking.

t. 33. **Cleaning Up.** The Contractor shall at all times keep the premises rom accumulations of waste material or rubbish caused by his employees rk and at the completion of the work he shall remove all his rubbish from bout the building and all his tools, scaffolding and surplus materials, and leave his work "broom clean" or its equivalent, unless more exactly ied. In case of dispute the Owner may remove the rubbish and charge st to the several contractors as the Architect shall determine to be just.

t. 34. **Cutting, Patching and Digging.** The Contractor shall do all g, fitting, or patching of his work that may be required to make its several come together properly and fit it to receive or be received by work of other ctors shown upon, or reasonably implied by, the Drawings and Specifica- for the completed structure, and he shall make good after them, as the tect may direct. Any cost caused by defective or ill-timed work shall be by the party responsible therefor. The Contractor shall not endanger ork by cutting, digging, or otherwise and shall not cut or alter the work other contractor, save with the consent of the Architect

. 35. **Delays.** If the Contractor is delayed in the completion of the work

by any act or neglect of the Owner or the Architect, or of any employee of
or by any other contractor employed by the Owner, or by changes ordered in
work, or by strikes, lockouts, fire, unusual delay by common carriers, una
able casualties, or any causes beyond the Contractor's control, or by d
authorized by the Architect pending arbitration, or by any cause which
Architect shall decide to justify the delay, then the time of completion
be extended for such reasonable time as the Architect may decide. No
extension shall be made for delay occurring more than seven days before
therefor is made in writing to the Architect. In the case of a continuing
of delay, only one claim is necessary. If no schedule is made under Ar
no claim for delay shall be allowed on account of failure to furnish dra
until two weeks after demand for such drawings and not then unless
claim be reasonable. This article does not exclude the recovery of dam
for delay by either party under Article 39 or other provisions in the Co
Documents.

Art. 36. Owner's Right to Do Work. If the Contractor should ne
to prosecute the work properly or fail to perform any provision of this cont
the Owner, after three-days' written notice to the Contractor, may, with
prejudice to any other remedy he may have, make good such deficiencies
may deduct the cost thereof from the payment then or thereafter due
Contractor; provided, however, that the Architect shall approve both s
action and the amount charged to the Contractor.

Art. 37. Owner's Right to Terminate Contract. If the Contractor sh
be adjudged a bankrupt, or if he should make a general assignment for the be
of his creditors, or if a receiver should be appointed on account of his insolve
or if he should, except in cases recited in Article 35, persistently or repeat
refuse or fail to supply enough properly skilled workmen or proper material
if he should fail to make prompt payment to subcontractors or for materi
labor, or persistently disregard laws, ordinances or the instructions of the Ar
tect, or otherwise be guilty of a substantial violation of any provision of the
tract, then the Owner, upon the certificate of the Architect that sufficient c
exists to justify such action, may, without prejudice to any other right or re
and after giving the Contractor seven-days' written notice, terminate
employment of the Contractor and take possession of the premises and
materials, tools and appliances thereon and finish the work by whatever me
he may deem expedient. In such case the Contractor shall not be entitl
receive any further payment until the work is finished. If the unpaid bal
of the contract price shall exceed the expense of finishing the work, incl
compensation to the Architect for his additional services, such excess sh
paid to the Contractor. If such expense shall exceed such unpaid balance,
Contractor shall pay the difference to the Owner. The expense incurred
the Owner as herein provided, and the damage incurred through the C
tractor's default, shall be certified by the Architect.

Art. 38. Contractor's Right to Stop Work or Terminate Contract.
the work should be stopped under an order of any court, or other p
authority, for a period of three months, through no act or fault of the C
tractor or of any one employed by him, or if the Owner should fail to p
the Contractor, within seven days of its maturity and presentation, any
certified by the Architect or awarded by arbitrators, then the Contractor
upon three-days' written notice to the Owner and the Architect, stop wo
terminate this contract and recover from the Owner payment for all work
cuted and any loss sustained upon any plant or material and reasonable p
and damages.

t. 39. **Damages.** If either party to this contract should suffer damage in ıanner because of any wrongful act or neglect of the other party or y one employed by him, then he shall be reimbursed by the other party ıch damage. Claims under this clause shall be made in writing to the liable within a reasonable time of the first observance of such damage ıot later than the time of final payment, except in case of claims under e 16, and shall be adjusted by agreement or arbitration.

. 40. **Mutual Responsibility of Contractors.** Should the Contractor damage to any other contractor on the work, the Contractor agrees, upon ıotice, to settle with such person by agreement or arbitration, if he o settle. If such other contractor sues the Owner on account of lamage alleged to have been so sustained, the Owner shall notify the actor, who shall defend such proceedings at the Owner's expense and, judgment against the Owner arise therefrom, the Contractor shall pay isfy it and pay all costs incurred by the Owner.

t. 41. **Separate Contracts.** The Owner reserves the right to let other ıcts in connection with this work. The Contractor shall afford other con- ırs reasonable opportunity for the introduction and storage of their mate- ınd the execution of their work and shall properly connect and coordinate ›rk with theirs. If any part of the Contractor's work depends for proper tion or results upon the work of any other contractor, the Contractor shall :t and promptly report to the Architect any defects in such work that r it unsuitable for such proper execution and results. His failure so to :t and report shall constitute an acceptance of the other contractor's work and proper for the reception of his work, except as to defects which may ›p in the other contractor's work after the execution of his work. To the proper execution of his subsequent work the Contractor shall measure already in place and shall at once report to the Architect any discrepancy en the executed work and the drawings.

. 42. **Assignment.** Neither party to the Contract shall assign the con- without the written consent of the other, nor shall the Contractor assign ıoneys due or to become due to him hereunder, without the previous n consent of the Owner.

. 43. **Subcontracts.** The Contractor shall, as soon as practicable after gning of the contract, notify the Architect in writing of the names of ntractors proposed for the principal parts of the work and for such others e Architect may direct and shall not employ any that the Architect may ı a reasonable time object to as incompetent or unfit. If the Contractor ıbmitted, before signing the contract, a list of subcontractors and the e of any name on such list is required or permitted after signature of nent, the contract price shall be increased or diminished by the difference en the two bids. The Architect shall, on request, furnish to any subcon- r, wherever practicable, evidence of the amounts certified to on his ıt. The Contractor agrees that he is as fully responsible to the Owner e acts or omissions of his subcontractors and of persons either directly or ctly employed by them, as he is for the acts and omissions of persons di- employed by him. Nothing contained in the Contract Documents shall any contractual relation between any subcontractor and the Owner.

. 44. **Relations of Contractor and Subcontractor.** The Contractor to bind every subcontractor and every subcontractor agrees to be bound, e terms of the General Conditions, Drawings and Specifications, as far as able to his work, including the following provisions of this Article, unless

specifically noted to the contrary in a subcontract approved in writ adequate by the Owner or Architect. This does not apply to minor contracts.

The Subcontractor agrees:

(a) To be bound to the Contractor by the terms of the General Cond Drawings and Specifications and to assume toward him all the obligation responsibilities that he, by those documents, assumes toward the Owner.

(b) To submit to the Contractor applications for payment in such reas time as to enable the Contractor to apply for payment under Article 26 of General Conditions.

(c) To make all claims for extras, for extensions of time and for damag delays or otherwise, to the Contractor in the manner provided in the Ge Conditions for like claims by the Contractor upon the Owner, except that time for making claims for extra cost as under Article 25 of the General Co tions is one week.

The Contractor agrees:

(d) To be bound to the Subcontractor by all the obligations that the Ow assumes to the Contractor under the General Conditions, Drawings and Spe cations and by all the provisions thereof affording remedies and redress to Contractor from the Owner.

(e) To pay the Subcontractor, upon the issuance of certificates, if issued u the schedule of values described in Article 26 of the General Conditions, amount allowed to the Contractor on account of the Subcontractor's wor the extent of the Subcontractor's interest therein.

(f) To pay the Subcontractor, upon the issuance of certificates, if issued ot wise than as in (e), so that at all times his total payments shall be as larg proportion to the value of the work done by him as the total amount certifie the Contractor is to the value of the work done by him.

(g) To pay the Subcontractor to such extent as may be provided by Contract Documents or the subcontract, if either of these provides for ea or larger payments than the above.

(h) To pay the Subcontractor on demand for his work or materials as fa executed and fixed in place, less the retained percentage, at the time the ce cate should issue, even though the Architect fails to issue it for any caus the fault of the Subcontractor.

(j) To pay the Subcontractor a just share of any fire-insurance money rec by him, the Contractor, under Article 21 of the General Conditions.

(k) To make no demand for liquidated damages or penalty for delay in sum in excess of such amount as may be specifically named in the subcont

(l) That no claim for services rendered or materials furnished by the C tractor to the Subcontractor shall be valid unless written notice thereof is g by the Contractor to the Subcontractor during the first ten days of the cale month following that in which the claim originated.

(m) To give the Subcontractor an opportunity to be present and to su evidence in any arbitration involving his rights.

(n) To name as arbitrator under Article 45 of the General Conditions person nominated by the Subcontractor, if the sole cause of dispute is the w materials, rights, or responsibilities of the Subcontractor; or, if of the S contractor and any other subcontractor jointly, to name as such arbitrator person upon whom they agree.

The Contractor and the Subcontractor agree that:

(o) In the matter of arbitration, their rights and obligations and all proce shall be analogous to those set forth in Article 45 of the General Conditio

Nothing in this Article shall create any obligation on the part of the Owner pay to or to see to the payment of any sums to any Subcontractor.

Art. 45. Arbitration. Subject to the provisions of Article 10, all questions dispute under this contract shall be submitted to arbitration at the choice either party to the dispute. The Contractor agrees to push the work vigor-sly during arbitration proceedings. The demand for arbitration shall be d in writing with the Architect, in the case of an appeal from his decision, hin ten days of its receipt and in any other case within a reasonable time er cause thereof and in no case later than the time of final payment, except to questions arising under Article 16. If the Architect fails to make a ision within a reasonable time, an appeal to arbitration may be taken as his decision had been rendered against the party appealing. No one shall nominated or act as an arbitrator who is any way financially interested this contract or in the business affairs of either the Owner, Contractor or chitect. The general procedure shall conform to the laws of the State in ich the work is to be erected. Unless otherwise provided by such laws, parties may agree upon one arbitrator; otherwise there shall be three, named, in writing, by each party to this contract, to the other party and the Architect, and the third chosen by these two arbitrators, or if they fail select a third within ten days, then he shall be chosen by the presiding officer the Bar Association nearest to the location of the work. Should the party nanding arbitration fail to name an arbitrator within ten days of his demand right to arbitration shall lapse. Should the other party fail to choose an itrator within said ten days, then such presiding officer shall appoint such itrator. Should either party refuse or neglect to supply the arbitrators with papers or information demanded in writing, the arbitrators are empowered both parties to proceed ex parte. The arbitrators shall act with promptness. here be one arbitrator his decision shall be binding; if three the decision any two shall be binding. Such decision shall be a condition precedent to right of legal action, and wherever permitted by law it may be filed in urt to carry it into effect. The arbitrators, if they deem that the case nands it, are authorized to award to the party whose contention is sustained h sums as they shall deem proper for the time, expense and trouble incident the appeal and, if the appeal was taken without reasonable cause, damages delay. The arbitrators shall fix their own compensation, unless otherwise vided by agreement, and shall assess the costs and charges of the arbitration n either or both parties. The award of the arbitrators must be in writing, , if in writing, it shall not be open to objection on account of the form of proceedings or the award, unless otherwise provided by the laws of the te in which the work is to be erected. In the event of such laws providing any matter covered by this article otherwise than as hereinbefore specified, method of procedure throughout and the legal effect of the award shall wholly in accordance with the said State laws, it being intended hereby to down a principle of action to be followed, leaving its local application to be pted to the legal requirements of the place in which the work is to be erected.

B. THE STANDARD FORM OF BOND *

USE IN CONNECTION WITH THE THIRD EDITION OF THE STANDARD FORM OF AGREEMENT AND GENERAL CONDITIONS OF THE CONTRACT

his form has been approved by the National Association of Builders' Exchanges, The onal Association of Master Plumbers, the National Association of Sheet Metal

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Contractors of the United States, the National Electrical Contractors' Associatio the United States, the National Association of Marble Dealers, the Building Gra Quarries Association, the Building Trades Employers' Association of the City of N York, and the Heating and Piping Contractors' National Association.

KNOW ALL MEN: That we..

(Here insert the name and address or legal title of the Contractor.)

............................(Two blank lines)*............................

hereinafter called the Principal, and..

(Here insert the name and address or legal title of one or more sureties.)

............................(Two blank lines)............................ a

............................(Two blank lines)............................ a

hereinafter called the Surety or Sureties, are held and firmly bound unto. ..

(Here insert the name and address or legal title of the Owner.)

............................(Two blank lines)............................

hereinafter called the Owner, in the sum of..

............................(Two blank lines)................(\$..........

for the payment whereof of the Principal and the Surety or Sureties bind the selves, their heirs, executors, administrators, successors and assigns jointly a severally, firmly, by these presents.

Whereas, the Principal has, by means of a written Agreement, dated........ ....entered into a contract with the Owner f

.... (Two blank lines) ....

a copy of which Agreement is by reference made a part hereof:

Now, Therefore, the Condition of this Obligation is such that if the Princip shall faithfully perform the Contract on his part, and satisfy all claims a demands, incurred for the same, and shall fully indemnify and save harmless t Owner from all cost and damage which he may suffer by reason of failure so do, and shall fully reimburse and repay the Owner all outlay and expense whi the Owner may incur in making good any such default, and shall pay all pe sons who have contracts directly with the Principal for labor or materials, th this obligation shall be null and void; otherwise it shall remain in full fo and effect.

Provided, however, that no suit, action or proceeding by reason of any defa whatever shall be brought on this bond after..................months fro the day on which the final payment under the Contract falls due.

And Provided, that any alterations which may be made in the terms of t Contract, or in the work to be done under it, or the giving by the Owner of a extension of time for the performance of the Contract, or any other forbeara on the part of either the Owner or the Principal to the other shall not in any w release the Principal and the Surety or Sureties, or either or any of them, th heirs, executors, administrators, successors, or assigns from their liability he under, notice to the Surety or Sureties of any such alteration, extension, or fo bearance being hereby waived.

Signed and Sealed this..............day of........................ 19

In Presence of

....		............................SEA
(Repeated three times)	as to	(Repeated three times)
....		....

* Dotted lines, as indicated, are in the standard documents and are omitted her ...ve space.

THE STANDARD FORM OF AGREEMENT BETWEEN CONTRACTOR AND SUBCONTRACTOR *

R USE IN CONNECTION WITH THE THIRD EDITION OF THE STANDARD FORM OF AGREEMENT AND GENERAL CONDITIONS OF THE CONTRACT

his form has been approved by the National Association of Builders' Exchanges, The ional Association of Master Plumbers, the National Association of Sheet Metal tractors of the United States, the National Electrical Contractors' Association of United States, the National Association of Marble Dealers, the Building Granite rries Association, the Building Trades Employers' Association of the City of New k, and the Heating and Piping Contractors' National Association.

THIS AGREEMENT, made this.............day of.............19..
and between.............................hereinafter called
Subcontractor and.............................
einafter called the Contractor.

TNESSETH, That the Subcontractor and Contractor for the considera- s hereinafter named agree as follows:

Section 1. The Subcontractor agrees to furnish all material and perform all k as described in Section 2 hereof for.....(Here name the kind of building)....
..............................(Blank lines)..............................
..................(Here insert the name of the Owner)..................
..............................(Blank lines)..............................
einafter called the Owner, at........(Here insert the location of the work.)....
..............................(Blank lines)..............................
ccordance with the General Conditions of the Contract between the Owner the Contractor, and in accordance with the Drawings and the Specifications pared by.......................hereinafter called the Architect, all of ch General Conditions, Drawings and Specifications signed by the parties reto or identified by the Architect, form a part of a Contract between the tractor and the Owner dated.............19.. and hereby become a t of this Contract.

Section 2. The Subcontractor and the Contractor agree that the materials e furnished and work to be done by the Subcontractor are (Here insert a ise description of the work, preferably by reference to the numbers of the Drawings the pages of the Specifications.)..............................
..............................(Blank lines)..............................

Section 3. The Subcontractor agrees to complete the several portions and whole of the work herein sublet by the time or times following:...........
.(Here insert the dates or date and if there be liquidated damages state them.)....
..............................(Blank lines)..............................

Section 4. The Contractor agrees to pay the Subcontractor for the perform- e of his work the sum of ....(Blank line)...........($...........) current funds, subject to additions and deductions for changes as may be eed upon, and to make payments on account thereof in accordance with tion 5 hereof.

Section 5. The Contractor and Subcontractor agree to be bound by the ms of the General Conditions, Drawings and Specifications as far as applicable this subcontract, and also by the following provisions:†

* Published by permission of The American Institute of Architects.

† Article 44 of the General Conditions of the Contract is here repeated in full with the eption of references to other articles. See page 1761.

Section 6..

........................(One page of blank lines) ....

Finally. The Subcontractor and Contractor, for themselves, their h successors, executors, administrators and assigns, do hereby agree to the performance of the covenants herein contained.

IN WITNESS WHEREOF they have hereunto set their hands the day date first above written.

In Presence of

....

Subcontract

....

Contra

D. STANDARD FORM OF ACCEPTANCE OF SUBCONTRACTOR PROPOSAL *

FOR USE IN CONNECTION WITH THE THIRD EDITION OF THE STANDARD FORM AGREEMENT AND GENERAL CONDITIONS OF THE CONTRACT

This form has been approved by the National Association of Builders' Exchanges, Th National Association of Master Plumbers, the National Association of Sheet Met Contractors of the United States, the National Electrical Contractors' Associatio the United States, the National Association of Marble Dealers, the Building Gran Quarries Association, the Building Trades Employers' Association of the City of N York, and the Heating and Piping Contractors' National Association.

Dear Sir: Having entered into a contract with (Here insert the name and a dress or corporate title of the Owner.)..

..................................(Blank line)..................................

for the erection of (Here insert the kind of work and the place at which it is to b erected.)..

..................................(Blank line)..................................

in accordance with plans and specifications prepared by (Here insert the na and address of the Architect.)..

..................................(Blank line)..................................

and in accordance with the General Conditions of the Contract prefixed to th specifications, the undersigned hereby accepts your proposal of (Here insert da

....

to provide all the materials and do all the work of (Here insert the kind of wo to be done, as plumbing, roofing, etc., accurately describing by number, page, etc., th drawings and specifications governing such work.)..

..................................(Blank lines)..................................

The Undersigned agrees to pay you in current funds for the faithful perform ance of the subcontract established by this acceptance of your proposal the su of..(\$..........

Our relations in respect of this subcontract are to be governed by the pla and specifications named above, by the General Conditions of the Contract far as applicable to the work thus sublet and especially by Article 44 of thos conditions printed on the reverse hereof.†

Very truly yours,

*Published by permission of The American Institute of Architects.

† Article 44 of the General Conditions of the Contract is printed in full on the reve side of the Institute's standard form. See page 1761.

he Subcontractor entering into this agreement should be sure that not
ely the above Article 44, but the full text of the General Conditions of the
ntract as signed by the Owner and Contractor is known to him, since such
text, though not herein repeated, is binding on him.

FFICIAL INSTITUTE DOCUMENTS OF A PERMANENT NATURE PUBLISHED (1922) BY THE AMERICAN INSTITUTE OF ARCHITECTS. TITLES AND PRICES

e Journal of the American Institute of Architects, monthly, 50 cts; yearly, to A. I. A. members	$2.50
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nograph on the Octagon (Thirty Drawings, 12×18, Photos and Text	12.50
e Standard Contract Documents:	
Agreement and General Conditions in Cover	.20
General Conditions without Agreement	.14
Agreement without General Conditions	.05
Bond of Suretyship	.03
Form of Subcontract	.04
Letter of Acceptance of Subcontractor's Proposal	.03
Cover (heavy paper, with valuable notes)	.01
Complete set in Cover	.30
m of Agreement, Owner and Architect (Percentage Basis)	.05
cular of Information on the Fee-Plus-Cost System of Charges (Explains Owner-Architect Agreement, Fee-Plus-Cost Basis)	.03
m of Agreement, Owner and Architect (Fee-Plus-Cost Basis)	.05
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ceedings of the Convention	Free
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REGULATION OF THE PRACTICE OF ARCHITEC TURE BY STATE LEGISLATION *

States Having Registration Laws (1923). Twenty-three have regist tion or license laws (1923) for the practice of architecture: California, Colora Florida, Georgia, Idaho, Illinois, Louisiana, Michigan, Minnesota, Montana, Ne Jersey, New York, North Carolina, North Dakota, Oregon, Pennsylvania, Sou Carolina, Tennessee, Utah, Virginia, Washington, West Virginia, and Wiscons

Laws are pending (1923) in Indiana, Iowa, Kansas, and other states.

Theory of Registration Laws. The reason for the REGULATION OF ARCI TECTURAL PRACTICE BY LAW is the fact that men improperly qualified practice can otherwise, at will, assume the title of ARCHITECT and impose u the public, thereby discrediting the profession. In some States and in Ca it seems evident that legislation was enacted for protection of local archit against encroachment on the part of non-resident architects. Such a mot is unworthy of the profession, whose efforts through legislation should be encourage design of higher artistic quality and to insure safe construct Some laws, like the first one formulated (Illinois 1897), are LICENSE LAWS in they tax every architect. Other laws, that in New York, for example, a REGISTRATION LAWS, undertake to issue certificates only to those qualified practice. Registration laws should not in a retroactive way attempt to depr those of their right who, by virtue of having been in bona-fide practice the law was enacted, have the legal right to continue in such practice, sub to the effect of public sentiment which may be created against non-regis architects, and subject also to responsibilities imposed by building ordina requiring safe construction. The THEORY OF THE REGISTRATION LAW is an architect should attain to a certain minimum general education, a cert minimum technical education, plus a certain minimum of practical experie before beginning practice on his own account. That theory is carried effect by requiring under penalty that no person may assume the title ARCH whether he is a new practitioner, or an experienced practitioner from wit the State, without first establishing his qualifications and receiving a certi authorizing him to use that title. The NEW YORK STATE LAW, printed

* This matter was prepared by D. Everett Waid, President (1922) of the New Y State Board of Examiners and Registration of Architects.

th, is typical of recent laws which attempt to embody this theory. References re are made to this law and the notation also that the American Institute Architects is prepared to cooperate with any persons interested who desire improve upon the laws already passed when trying to secure in other States ch legislation as will tend to raise the standard of qualifications of architects. ch legislation will certainly achieve its highest end if looked upon as educanal in purpose. Incidentally it may be remarked that the best interests of will be conserved if earnest efforts are made toward a COMMON STANDARD ich will encourage reciprocal relations among the States. An architect o has demonstrated his qualifications by passing a proper examination in e State should not be harassed by repetitions of the test in other States which he may choose to practice.

Standards of Education. The GENERAL EDUCATION required under the w York law is the completion of a high-school course, or equivalent thereof; o the satisfactory completion of such courses in mathematics, history, and e modern language, as are included in two years of an approved institution nting the degree of A.B. Five years' PRACTICAL EXPERIENCE in the office a reputable architect, beginning after the high-school course, is required ore a candidate can take the TECHNICAL EXAMINATION conducted by the ard of Examiners. This technical examination is not required of graduates recognized schools of architecture who shall have had, also, three years' pracıl experience.

dministration of Registration Laws. In New York State, architects admitted to practice by the Regents of the University of the State, who ninister the law through a BOARD OF EXAMINERS AND REGISTRATION OF CHITECTS. In other States, the Boards of Examiners are appointed by the vernors.

pplication for Certificates. Application-blanks and information regard- admittance to practice, dates of examinations, etc., can be obtained by ressing the Board of Examiners and Registration of Architects, Education lding, Albany, N. Y. In other States, such inquiries may be addressed he Secretary of State.

Iodel Registration Law. Those interested in STATE LEGISLATION REGU- ING THE PRACTICE OF ARCHITECTURE AND THE EDUCATION OF ARCHITECTS secure copies of a bill issued to serve as a BASIC MODEL which, with suitable lifications, may be enacted in any State. Address the Secretary, American itute of Architects, The Octagon House, Washington, D. C.

GISTRATION OF ARCHITECTS IN THE STATE OF NEW YORK *

law in relation to the practice of architecture and the rules of the State Board of Examiners and Registration of Architects approved by the Regents of The University of the State of New York

he Assistant Commissioner and Director of Professional Education is in ge of universities, colleges, professional and technical schools, the execution he laws concerning the professions, and the relations and chartering of tutions. All correspondence relating to the issuance of certificates, the ils of licensing examinations, and admissions to the practice of architecture

*The form of the law itself and of the State official documents, with the exception of ype, are inserted as enacted and printed, without further editing.

should be addressed to the Director of the Examinations and Inspecti Division, Albany, N. Y.

REGISTERED ARCHITECTS

General business law (L. 1909, Ch. 25) Chapter 20 of the consolidated la became a law February 17, 1909

Article 7-A, Registered Architects

Became a law April 28, 1915 (Laws of 1915, Chapter 454). As amended by Laws of 19 Chapter 77.

Section 77. Registered Architects.
Section 78. Board of Examiners.
Section 79. Qualifications. Examinations. Fees.
Section 79a. Certificates.
Section 79b. Violation of Article.

77. **Registered Architects.** Any person residing in or having a place of business in the State, who, before this article takes effect, shall not have been engaged in the practice of architecture in New York State, under the title of architect, shall, before being styled or known as an architect, secure a certificate of his qualification to practice under the title of architect, as provided by this article. Any person who shall have been engaged in the practice of architecture under the title of architect, before this article takes effect, may secure such certificate, in the manner provided by this article. Any person having a certificate pursuant to this article may be styled or known as a registered architect. No other person shall assume such title or use the abbreviation R. A., or any other words, letters or figures to indicate that the person using the same is a registered architect; but this article shall not be construed to prevent persons other than architects from filing applications for building permits or obtaining such permits.

78. **Board of Examiners and Registration.** There shall be a State Board of Examiners and Registration of Architects, who, and their successors, shall be appointed by and hold during the pleasure of the Board of Regents of the University of the State, and who, subject to the approval and to the rules of the Regents, may make rules for the examination and registration of candidates for the certificates provided for by this article. Such board of examiners shall be composed of five architects, who have been in active practice in the State of New York for not less than ten years, previous to their appointment, selected by the Regents. Such examiners shall be entitled to such compensation for their services under this article as the Board of Regents shall determine, not exceeding in the aggregate the amount of fees collected from applicants for certificates.

79. **Qualification. Examination. Fees.** Any citizen of the United States, or any person who has duly declared his intention of becoming a citizen, being at least twenty-one years of age and of good moral character, may apply for examination or certificate of registration under this article, but before securing such certificate shall submit satisfactory evidence of having satisfactorily completed the course in high school approved by the Regents of the University or the equivalent thereof and subsequent thereto of having satisfactorily completed such courses in mathematics, history and one modern language, as are included in the first two years in an institution approved by the Regents, conferring the degree of bachelor of arts. Such candidate shall

dition submit satisfactory evidence of at least five years' practical experience *the* office or offices of a reputable architect or architects, commencing after completion of the high school course. The board of examiners may accept isfactory diplomas or certificates from approved institutions covering the irse required for examination. Upon complying with the above require-nts, the applicant shall satisfactorily pass an examination in such technical l professional courses as are established by the board of examiners. The rd of examiners in lieu of examinations may accept satisfactory evidence any one of the qualifications set forth under subdivisions 1 and 2 of this tion.

. A diploma of graduation or satisfactory certificate from an architectural ege or school approved by the Regents, together with at least three years' ctical experience in the office or offices of a reputable architect or architects; the three years' experience shall be counted only as beginning at the com-ion of the course leading to the diploma or certificate; the State Board of aminers and Registration of Architects may require applicants under this division to furnish satisfactory evidence of knowledge of professional prac-;

. Registration or certification as an architect in another state or country, ere the qualifications required for the same are equal to those required in State;

. The board of examiners in lieu of all examinations shall accept satisfactory lence as to the applicant's character, competency and qualifications, and he has been continuously engaged in the practice of architecture for more n two years next prior to the date when this article shall take effect, or that ias been actually engaged in the practice of architecture on his own account s a member of a reputable firm or association for more than one year prior he date when this article shall take effect; providing the application for certification shall be made on or before January 1, 1918. Any architect has lawfully practiced architecture for a period of more than ten years iout the State shall be required to take only a practical examination, which l be of the nature to be determined by the State Board of Examiners and istration of Architects. Every person applying for examination or certifi- of registration under this article shall pay a fee of twenty-five dollars to Board of Regents.

a. Certificates. 1. The result of every examination, or other evidence ualification, as provided by this article, shall be reported to the Board of ents by the board of examiners, and a record of the same shall be kept he Board of Regents, and such board shall, unless deemed otherwise advis-, issue a certificate of registration to every person certified by the board aminers as having passed such examination or as being otherwise qualified e entitled to receive the same.

Every person securing such certificates shall register in the office of the ity clerk of the county in which he maintains a place of business, in a book by the clerk for such purpose, his name, residence, place and date of birth post office address, source, number and date of his certificate of registration ractice architecture and the date of such registration, which registration iall be entitled to make only upon showing to the county clerk his certificate gistration and making an affidavit of the above facts, and that he is the tical person named in the certificate; that before receiving the same he plied with all of the preliminary requirements of this article and the rules e Regents and the board of examiners as to the terms and the amount udy and examination; that no money other than the fees prescribed by

this article and such rules was paid directly or indirectly for such registr
and that no fraud, misrepresentation or mistake in material regard was
ployed or occurred in order that such certificate should be made, which and
shall be preserved in a bound volume by the county clerk. The county
shall indorse or stamp on the back of the certificate the date and his
preceded by the words "registered as authority to practice as a regis
architect, in the clerk's office of......................county"; and
issue to each person duly registering and making such affidavit a certific
of registration in his county which shall include a transcript of the registra
Such transcript and the certificate of registration may be offered as pres
tive evidence in all courts of the facts stated therein. The county cl
fee for taking such registration and affidavit and issuing such certificate s
be one dollar. Any person who, having lawfully registered as aforesaid, sh
thereafter change his name in any lawful manner, shall register the new na
with a marginal note of the former name, and shall note upon the margin
the former registration the fact of such change and a cross-reference to the
registration. A county clerk who knowingly shall make or suffer to be m
upon the book of registry of architects kept in his office any other entry th
is provided for in this article shall be guilty of a misdemeanor.

3. An architect having duly registered to practice as a registered architect
one county and removing his practice or a part thereof to another county
regularly engaging in practice or opening an office in another county, shall
or send, by registered mail, to the clerk of such other county, his certific
of registration. If such certificate clearly shows that the original registra
was duly issued under seal by the Board of Regents, the clerk shall thereup
register the applicant in the latter county on receipt of a fee of 25 cents
shall stamp or indorse on such certificate the date and his name, preceded
the words "Registered also in......................county," and retu
the certificate to the applicant.

4. The Board of Regents may revoke any certificate, if such action be reco
mended by the board of examiners, after thirty days' written notice to
holder thereof and after a hearing before the board of examiners, upon p
that such certificate has been obtained by fraud or misrepresentation, or
proof that the holder of such certificate has been guilty of felony in connecti
with the practice of architecture.

79b. Violation of Article. Any violation of this article shall be a
demeanor, punishable for the first offense by a fine of not less than fifty
not more than one hundred dollars, and for a subsequent offense by a
of not less than two hundred nor more than five hundred dollars, or impri
ment for not more than one year, or both.

SYNOPSIS OF SUBJECTS ON WHICH EXAMINATIONS ARE BASE

The examinations of applicants for certificates shall be based on the
following subjects or groups:

a. History of Architecture. The candidate shall give evidence in
examination that he understands the essentials that give character to
various historic styles of architecture by clear descriptive analyses of
construction, general expression and ornament. Questions will be a
relating to:

(1) Architecture in various countries.

* Taken from the Rules of the New York State Board of Examiners and Regist
of Architects.

(2) Styles and orders. Sketches and names of examples.
(3) Sculpture and painting and color as applied to architecture.
(4) Furniture and decoration.

). **Architectural Composition.** The candidate shall give evidence that understands the broad principles underlying the subject of architectural nning by the application of the same to specific problems stated in the mination. The social, economic and physical requirements of several hitectural problems will be outlined and the candidate will be asked to te the principal considerations that would guide him in the choice of an angement of plan that would most adequately express and fulfill the con- ions suggested. Small sketches will be required to illustrate the application the principles involved. Questions will refer to:

1) Principles of Planning: Problems in planning individual buildings, ups and towns; illustrations may be asked to show how plans may be uenced by considerations, esthetic, structural, and as to kinds of materials, l modifications of plan due to considerations of occupancy and of fire pre- tion both for fireproof and non-fireproof buildings.

2) Esthetic Design: Original examples will be required illustrating prin- es involved in the solution of practical problems and the relation of plan elevation.

. **Architectural Engineering.** In this examination the candidate shall evidence that he has a thorough understanding of the appropriate use the various materials used in buildings. He will be required also to solve ain technical problems such as the calculation of the proper economic ensions of various structural members common to buildings, in the several erials noted. Candidates will not be required to make complicated calcu- ons, and the use of handbooks will be permitted. Questions will be asked o the knowledge of these subjects that an architect should possess in order perly to advise his clients and to design or to direct the designing of suitable hanical equipment for buildings of different classes. Questions will be asked ting to:

) Structural Design:
olumn, girder, joist and truss designs.
Vind bracing for buildings of different classes.
arious types of foundations and conditions under which their use is advisable.
arious kinds of bottom met with in ordinary practice and unit loads allowable er foundations in each case.
ifferent types of concrete floor slab construction in common use.
tructural design as affected by fire and resistive qualities of different build- materials.

) Use of Materials:
trength of materials, durability and considerations of wear and repair.
sthetic reasons for use of different materials.

) Heating and Ventilating:
arious systems and reasons for and against use under specific conditions.
nportant features of design that should be specified.

) Electric Equipment: General questions rather than technical.
arious kinds of current and considerations involved.
inds of wiring and insulation; methods and materials.
ight distribution.
ighting fixtures; esthetic and practical design; important mechanical ils.

Generators, motors, storage batteries, and advice to clients regarding the same.

Independent power plant considerations.

(5) Plumbing and Fire Protection Equipment:

Supply and waste systems.

Kinds of material for piping and reasons for use.

Kinds of materials for fixtures.

Sewage disposal plants and considerations involved.

Water supply, different systems and considerations of supply and filtratio

Sprinklers and other fire protection equipment.

(6) Elevators:

Types of elevators.

Arrangement and location of elevators.

d. Architectural Practice. In this examination the candidate shall gi evidence that he understands the moral and the legal responsibilities of th architect in the proper performance of his duties as such. He will be requi to outline or draft clauses of contracts between owner and architect and state specifically the content of the clauses included in the contract betw owner and contractor which are incorporated for the purpose of defining t architect's authority and responsibility to both parties of the contract. Ea candidate will be required to show that he understands the major provisi of state, county and municipal building laws and ordinances and how the s affect the different classes of buildings. He shall be able also to cite the co petent authority under whose jurisdiction permits for the erection and occupa of various types of buildings must be obtained. Questions will be asked rel ing to:

(1) Business and Professional Functions of Architects:

Professional relation of clients and contractors.

Essential provisions in contract between architect and his client.

When the architect is disinterested arbitrator, and when he properly may a as an agent.

Relation of architects to each other in ordinary practice, in association, a in consultation, and when one architect displaces another on a given piece work.

Sources and kinds of compensation for architect's services.

Responsibilities of architects and methods of conducting their business.

Scope of architect's work, esthetic and structural.

When expert's services should be advised.

Scope of architect's work, administrative business, and legal contra arbitrations, court evidence, contractors in default, when counsel of clie lawyer should be advised.

(2) Building Laws:

State, county and municipal laws, ordinances and regulations, and h they affect different classes of buildings.

Filing drawings and specifications and obtaining permits.

(3) Contracts:

Drawings, specifications and agreement as essential parts of the custo contract between owner and builder.

Variations in kinds and forms of agreements and contracts.

Definition of architect's authority.

Provisions as to bids, letting contracts, unit prices, requisitions, certif and payments.

Insurance: fire, liability, compensation.

Bonding contractors.

(4) Specifications:

General conditions, purposes and scope.

Scope and purposes and limitations of general clauses.

Principles which should be observed in writing specifications.

Right and wrong methods of specifying qualities of materials and workman-ip.

(5) Drawings:

Purposes, use and limitations of preliminary drawings.

Essentials which should be embodied in contract drawings.

Purposes and limitations of detail and other working drawings which are t contract drawings.

REGISTRATION OF SCHOOLS OF ARCHITECTURE *

A school of architecture may be registered as maintaining a satisfactory indard and may be legally incorporated. Incorporation by the Regents will made on formal application and inspection by the Department which show at the school possesses the minimum requirements.

Application. An educational institution desiring admission to or incor-ration or registration by the University must file a written application giving information requested in the form prescribed by the Commissioner of ucation. A form will be mailed on application to the Assistant Commissioner Higher Education. Such application must be on file in the Education partment at least 10 days before the meeting of the Regents at which action reon is to be taken.

Accrediting. Institutions unable to meet the standards required by the gents for registration in full shall be accredited by the Department for one nore years of professional training as they meet the requirements for admission l for professional training set by the Regents standards.

Recognition Accorded Accredited Professional Schools. Professional ools registered by the Regents shall give the work of accredited institutions higher recognition than that accorded such institutions in the Department's redited list, viz.: (1) The successful completion of a four-year course in a fessional school accredited by the Department for three years shall be orded three years' recognition only; (2) the successful completion of a ee-year course in a professional school accredited by the Department for years shall be accorded two years' recognition only; (3) the successful npletion of a two-year course in a professional school accredited by the partment for one year shall be accorded one year's recognition only. A istered school may refuse to accord an accredited institution the recognition en it by the Department but it may not give it any higher recognition.

Comity of Action in the Transfer of Students from One Professional ool to Another. The Department does not consider a course in a school rchitecture satisfactory if more than two conditions, one major of 100 hours one minor of 50 hours, are allowed students for promotion from one year's s to the next.

*Taken from the Rules of the New York State Board of Examiners and Registration Architects. As schools may be added to the accredited lists, these lists must be sed from time to time.

REGENTS' RULES

Schools of Architecture *

440. Definitions. SCHOOL OF ARCHITECTURE means any college or sch of architecture, or school, department or course of architecture in a col or university, whatever the corporate title.

441. Requirements. A SCHOOL OF ARCHITECTURE, legally incorpora may be registered as maintaining proper standards. It must afford sa factory instruction in such technical and professional courses as are establis by the board of examiners, for admission to the examinations in the hist of architecture, architectural composition, architectural engineering and arc tectural practice.

442. General Education. A. PRELIMINARY. For admission to a sch of architecture, evidence shall be required showing the satisfactory complet of a four-year course in a secondary school approved by the Board of Regent or the equivalent, 72 counts in the academic examinations. B. HIGH For admission to the examinations for the certificate of R. A. evidence sh be required of such courses in mathematics, history and modern languages are included in the first two years of the curriculum leading to the degree bachelor in arts, or the equivalent, graduation from a junior college appro by the Board of Regents.

SCHOOLS OF THE UNITED STATES AND CANADA REGISTERED OR ACCREDITED.† JUNE, (1918)

Alphabetically Arranged by States

United States

California. REGISTERED: School of Architecture, University of Califor Berkeley. (Graduate course, one or two years.)

District of Columbia. REGISTERED: Department of Architecture, Geo Washington University, Washington. (Course, four years.)

Georgia. REGISTERED: Department of Architecture, Georgia School Technology, Atlanta. (Course, four years.)

Illinois. REGISTERED: Chicago School of Architecture, Armour Institute Technology, Chicago. (Course, four years.)

Department of Architecture, University of Illionis, Urbana. (Course, fo years.)

Indiana. REGISTERED: College of Architecture, University of Notre Dam Notre Dame. (Course, four years.)

Department of Architectural Engineering, Rose Polytechnic Institute, Ter Haute. (Course, four years.)

Kansas. Department of Architecture, Kansas State Agricultural Colleg Manhattan. (Course, four years.)

Department of Architectural Engineering, University of Kansas, Lawren (Course, four years.)

* Taken from the Rules of the New York State Board of Examiners and Registratio of Architects. As schools may be added to the accredited lists, these lists must b revised from time to time.

† This is the list of Schools or Departments of Architecture in the United States an Canada, registered or accredited by the New York State Board of Examiners and Regi tration of Architects, and must be added to from time to time.

Louisiana. REGISTERED: School of Architecture and Architectural Engi-
ering, Tulane University, New Orleans. (Courses, four years.)

Massachusetts. REGISTERED: Departments of Architecture and Archi-
tural Engineering, Massachusetts Institute of Technology, Cambridge.
ourses, four years.)

School of Architecture, Harvard University, Cambridge. (Graduate Course,
ee years.)

Michigan. REGISTERED: College of Architecture, University of Michigan,
n Arbor. (Course, four years.)

Minnesota. REGISTERED: Department of Architecture, University of
nnesota, Minneapolis. (Course, four years.)

Missouri. REGISTERED: School of Architecture, Washington University,
Louis. (Course, four years.)

Nebraska. REGISTERED: Department of Architectural Engineering, Uni-
sity of Nebraska, Lincoln. (Course, four years.)

New York. REGISTERED: College of Architecture, Cornell University,
aca. (Course, four or five years.)

Department of Architecture, Syracuse University, Syracuse. (Course, four
rs.)

School of Architecture, Columbia University, New York. (Course, four
rs.)

Ohio. REGISTERED: Department of Architecture, Ohio State University,
umbus. (Course, four years.)

Oklahoma. REGISTERED: Department of Architecture, Oklahoma Agri-
tural and Mechanical College, Stillwater. (Courses, four years.)

Pennsylvania. REGISTERED: Department of Architectural Engineering,
nsylvania State College, State College. (Course, four years.)

Department of Architecture, University of Pennsylvania, Philadelphia
ourses, four years.)

Department of Architecture, Carnegie Institute of Technology, Pittsburgh
ourse, four years.)

Texas. REGISTERED: Departments of Architecture and Architectural
gineering, Agricultural and Mechanical College of Texas, College Station.
ourses, four years.)

School of Architecture, University of Texas, Austin. (Course, four years.)

Canada

Ontario. REGISTERED: Department of Architecture, University of Toronto,
ronto. (Course, four years.)

Quebec. Department of Architecture, McGill University, Montreal.
ourse, five years.)

SYNOPSIS OF REGISTRATION LAWS *

This study † is made for those who would see at a glance the statutory
uirements for the practice of architecture throughout the United States.

* As States are added to the list of those which have laws for the Registration of
hitects, these lists must be revised from time to time. Georgia, Michigan, Penn-
rania, Virginia, and Washington have been added to the list, up to January, 1921.

† Taken from Handbook No. 35, published annually by The University of the State
New York, and containing information relating to the Registration of Architects.

There are four distinct lines of statutory requirements: (1) Preliminary e tion; (2) professional training; (3) licensing test; (4) registry. The items with (5) the title of the executive officer and the administrative are given uniformly in this synopsis.* If there are no statutory requirem the word "none" covers the item.

California. (1) None; (2) none; (3) examination; (4) with the recor of the county of residence annually; (5) secretary, State Board Architect San Francisco.

Colorado. (1) None; (2) none; (3) examination or certificate from similarly constituted board of another state; (4) with the secretary of state annually with the board; (5) Secretary, State Board of Examiners of Archi tects, Denver.

Idaho. (1) Approved high school course or its equivalent and in addition two-year course in English and mathematics such as is required in an approv B. A. course; (2) three years' practical experience in the office of a reputab architect; (3) examination or in lieu of all examinations, graduation from approved architectural school or registartion as an architect in another sta whose standard equals that of this board; (4) with the secretary of state; (Secretary, State Board of Examiners of Architects.

Illinois. (1) None; (2) none; (3) examination; (4) with the clerk of t county of practice, annually; (5) Secretary, Department of Registration a Examination, Springfield.

Louisiana. (1) Good primary education; (2) none; (3) examination diploma from an approved school of architecture; (4) with the district co clerk of the parish of residence and annually with the board; (5) Secreta Board of Architectural Examiners, New Orleans.

Montana. (1) None; (2) none; (3) examination or a license from anoth state board; (4) with the clerk and recorder of the county of residence a annually with the state treasurer; (5) Secretary, Board of Architectural Exam ners.

New Jersey. (1) None; (2) none; (3) examination or a license from similarly constituted board of another state or membership in the Americ Institute of Architects; (4) with the board, annually and with the secreta of state; (5) Secretary, State Board of Architects, Trenton.

New York. (1) Approved high school course or the equivalent and i addition such course in mathematics, history and one modern language as a included in an approved two-year B. A. course; (2) at least five years' practic experience in the office of a reputable architect; (3) examination or graduati from an approved architectural school with three years' experience or regi tration in another state or country having standards equal to that of this boar (4) with the Board of Regents; (5) Secretary, State Board of Examiners a Registration of Architects, New York.

North Carolina. (1) Prescribed by the board; (2) prescribed by t board; (3) examination or a certificate from a similarly constituted board i another state or membership in the American Institute of Architects; (with the clerk of the superior court of the county of residence; (5) Secretary the Board of Architectural Examination and Registration.

North Dakota. (1) Approved high school course or its equivalent; (three years' practical experience in the office of a reputable architect; (

* The names of the executive officers, Secretaries of the Boards, etc., are omitt here, as the personnel is constantly changing.

amination or a license from another state board whose standard equals that this board or membership in the American Institute of Architects; (4) with e secretary of state and annually with the board; Secretary, State Board of rchitecture, Bismarck.

South Carolina. (1) None; (2) at least two years' experience in archictural work; (3) examination or graduation from an approved school of chitecture; (4) with the board, annually; (5) Secretary, State Board of rchitectural Examiners, Columbia.

Utah. (1) None; (2) none; (3) examination; (4) with the board, annually;) Secretary, State Board of Architecture.

Wisconsin. (1) None; (2) at least five years' practical experience in the fice of a reputable architect; (3) examination or a satisfactory certificate om a recognized architectural school with three years' experience or registraon with the board of another state or country whose standards are not lowe an those of this board; (4) with the Industrial Commission; (5) Secretary the Board of Examiners of Architects, Madison.

DUCATIONAL INSTITUTIONS IN THE UNITED STATES AND CANADA OFFERING COURSES IN ARCHITECTURE. TRAVELLING FELLOWSHIPS AND SCHOLARSHIPS

1. Association of Collegiate Schools of Architecture

Association of Collegiate Schools of Architecture. Organized in 1912 advance the standards of architectural education in the United States. lembership (1921) represents fifteen architectural schools in Columbia, ornell, Harvard, Syracuse, Yale, and Washington Universities; the Universities f California, Illinois, Kansas, Michigan, Minnesota, Oregon and Pennsylvania, nd the Carnegie and Massachusetts Institutes of Technology. (Officers 1921: resident Emil Lorch; Vice-President, William Emerson; Secretary-Treasurer, larence A. Martin, Cornell University, Ithaca, N. Y. The Association equires for the admission of an institution to membership the attainment of ertain MINIMUM CONDITIONS in its course in architecture, a detailed list of which will be furnished upon application to the secretary. They are here roadly summarized as follows: (1) Collegiate rating under the Carnegie oundation; (2) A course of at least 120 CREDIT-HOURS of both general and rofessional studies of certain minimum range and approved method of presenation, leading to a degree not less than BACCALAUREATE; (3) Such character f staff and administration, standing of course and adequacy of equipment s will reasonably assure quality of performance; (4) A demonstration of sucess in operation through a period of at least four years. The Association olds an annual conference on architectural education to which representatives f all American schools are welcomed.

2. Educational Institutions, Fellowships, and Scholarships

Academy of Architecture and Industrial Science, St. Louis, Mo. This s a private school founded by Mr. Maack in 1885, and designed more paricularly to meet the wants of building tradesmen, offering them such instruction s is necessary to attain the highest proficiency in their trade and a thorough

understanding of the plans and details of complicated buildings. The also a special course for those desiring to fit themselves for positions as draug men in architects' offices. Tuition for the regular course is $50 for a th months' term, or $300 for the full course of eight terms, or $100 for the y Several special courses with varying tuition.

Alabama Polytechnic Institute, Auburn, Ala. DEPARTMENT ARCHITECTURE. (1) Full four-year course leading to the degree of Bach of Science in Architecture. (2) Full four-year course leading to the deg of Bachelor of Science in Architectural Engineering. (3) Two-year speci course for draftsmen and college graduates. Tuition free to residents Alabama; $20 per year for others. About two dozen LOAN-SCHOLARSHIPS $100 or more per annum. Limited number of FELLOWSHIPS of $250 for p graduates. Illustrated Announcement giving details, sent on request.

American Academy in Rome, Fellowship in Architecture. RO PRIZE. The fellowship is awarded annually and is of the value of $1 000 a y for three years. The award is made on competitions which are open only unmarried male citizens of the United States, who comply with the regulati of the Academy. Candidates are required to be (1) graduates of one of t architectural schools included in the accepted list of the Academy; or (2) gr uates of a college or university of high standing who hold certificates of at le two years' study in one of such architectural schools; or (3) Americans w are pupils of the first class of the School of Fine Arts at Paris, and who ha obtained at least three values in that class. There is no age-limit. Informati as to the terms and conditions of the competitions may be obtained from t Secretary of the Academy, 101 Park Avenue, New York City.

American School of Correspondence, Chicago, Ill. Correspondenc courses in Architecture, Architectural Engineering, Contracting and Building Reinforced Concrete, Architectural Design, and Structural Draughting. Bul tin sent on application.

Armour Institute of Technology, Chicago, Ill. Full four-year cou leading to the degree of Bachelor of Science in Architecture. Applicants f admission must have completed the regular four-year high-school cours A HOME TRAVELING-SCHOLARSHIP, four prizes, and a medal are awarded ann ally. Tuition, $180 per year.

Beaux Arts Institute of Design, 126 East 75th Street, New York, N.Y. DEPARTMENT OF ARCHITECTURE. (Address all communications to this depart ment.) The course, established in 1893, consists (1920) of a series of thirty-fi competitions, issued annually, for the study of architectural design and the sty of architecture, open to the draughtsmen and students in architectural schoo in the United States and Canada, and modeled on the system of instructi adopted by the École des Beaux Arts in Paris. The course is free, except fo the annual fee of $2 for registration of each student. There are no restrictio as to the age, nationality, or sex of the students. No preliminary examinatio are given, but new students are expected to have a knowledge of the five orde of architecture. BRONZE AND SILVER MEDALS are awarded for excellence i design and money-prizes are offered in special prizes for decoration, grou planning of buildings, etc. CERTIFICATES are presented to all students o Class A completing the course as defined in the circular of information, which is furnished on request. During the season 1917–1918 the work was carri on by one hundred and eleven correspondents of the Institute in eighty-eigh different cities, with a total of seven hundred and seventy-four students.

DEPARTMENT OF INTERIOR DECORATION (Address all communications to thi

partment): The course consists of programmes for competitions issued ery six weeks to those who apply for them. These may be executed by idents situated in any locality and sent in to the Institute where they will criticized and judged on fixed dates by a jury of experts. BRONZE AND VER MEDALS are awarded for excellence. An atelier for male students under instruction of several decorators exists in the building of the Institute. ere are no fees of any kind. No formalities or examinations are necessary admission to the atelier. A circular is furnished on request.

DEPARTMENT OF SCULPTURE. (Address all communications to this depart-nt.) Ateliers for male students for each one of the three courses (Archi-tural Ornament, Life Drawing, and Modeling and Composition) exist in building of the Institute. No examinations, formalities, or fees of any id. Open all day all the year round. Instructors visit their classes twice veek. Judgments by expert juries every four weeks on work of preceding nth. BRONZE AND SILVER MEDALS awarded. Circular furnished.

DEPARTMENT OF MURAL PAINTING. (Address all communications to this partment.) The course consists of problems, programmes of which are ied every month to those who apply for them. Judgments by a jury of ists every month on the designs handed in. BRONZE AND SILVER MEDALS arded. No examinations, formalities, or fees of any kind. No atelier this department at the Institute. Students work up their problems under ir own instructors wherever they may be situated. Circular furnished.

BEAUX ARTS ARCHITECTS, SOCIETY OF. 126 EAST 75TH STREET, NEW RK, N. Y. The course in Architectural Design established in 1893 and merly conducted by the Committee on Education of this Society, is now ried on by the Beaux Arts Institute of Design. (See Beaux Arts Institute Design.)

PARIS PRIZE. THIS SCHOLARSHIP-PRIZE is usually conducted annually by Society of Beaux Arts Architects. Under its conditions the winner re-ves $1 200, per annum for two years and a half, to study architecture in ris at the Ecole des Beaux Arts, into the upper class of which he is received thout further examinations. The competition beginning January 10th, 20, for this scholarship consisted of two preliminaries and one final com-tition and was open to all male citizens of the United States under thirty-two ars of age on July 1st, 1920. A circular is furnished on request.

Carnegie Institute of Technology, Pittsburgh, Pa. DIVISION OF THE TS; SCHOOL OF ARCHITECTURE. (1) A complete course in architecture for day-idents for which the degree of Bachelor of Architecture in Design is awarded those specializing in design and allied subjects (Option 1), and the degree of chelor of Architecture in Construction to those in construction and allied bjects (Option 2). From four to five years are required for the completion of escribed work. (2) For graduate day-students a course of advanced studies design and allied subjects, scheduled to cover one year, and leading to the gree of Master of Arts. (3) A partial day-course, scheduled to cover two ars, for experienced draughtsmen and designers, for which a certificate of oficiency is awarded. (4) A course for night-students for which a Certificate Proficiency is awarded. This course includes the same work as is required day-students in design, freehand drawing and modeling. Tuition: For day-hool, $75; for night-school, $20 per year.

Catholic University of America, Washington, D. C.

Clemson Agricultural College of South Carolina, Clemson College, S. C.

Columbia University, New York, N. Y. SCHOOL OF ARCHITECTURE.) Full four-year course leading to the degree of Bachelor of Architecture.

Receives only students with at least two years of college training. In connection with Columbia College, there is a six-year course giving the d of A.B., at the end of four years and B.Arch. at the end of six years. Advanced courses leading to the degree of Master of Science in Architec Tuition $6 per "tuition-point," totaling about $250 per year. There are th TRAVELING-FELLOWSHIPS, awarded as follows: One is available each y with a stipend of about $1 500; the McKIM FELLOWSHIP every third y beginning 1916–17; the SCHERMERHORN FELLOWSHIP, every third year, beg ning 1918–19; and the PERKINS FELLOWSHIP, every third year, begin 1920–21. Each of these requires the winner to devote one year to for travel and study.

EXTENSION-TEACHING, evening and afternoon courses. A course lead to a Certificate of Proficiency in Architecture is offered. This covers roug six years, depending on how much is taken each year. Equivalent of day-co in instruction. Tuition, $6 per "tuition-point," each course having a st point-value. Graduation accepted in lieu of examinations for state lice There are Special Students, also, under Extension-Teaching who select th own course of study in subjects for which they are qualified. All informati may be obtained from the Curator.

Cornell University, Ithaca, N. Y., COLLEGE OF ARCHITECTURE. (1) four-year general course in architecture, leading to the degree of Bach of Architecture, and a similar course with engineering electives, leading to th degree of Bachelor of Science in Architecture. (2) Five-year courses in arc tecture, the same as the above, but with additional work in the arts and scienc leading to the same degrees. (3) Six-year courses in arts and sciences an architecture, or in engineering and architecture, leading to the degrees of A and B.Arch., or C.E. and B.S.Arch. (4) A two-year special course in arc tecture, leading to a certificate. (5) Graduate courses in architecture, lead to the degree of Master of Architecture. Tuition, $200 a year.

George Washington University, Washington, D. C. DEPARTME OF ARTS AND SCIENCES. Course in Architecture. Four-year course in arc tecture, leading to the degree of Bachelor of Science in Architecture. Courses instruction open to qualified special students, without reference to any degr Full tuition $180; part-time students pay $6 for each semester-hour credit.

Georgia School of Technology, Atlanta, Ga. DEPARTMENT OF ARCH TECTURE. (1) Full four-year course leading to the degree of Bachelor Science in Architecture. (2) Two-year special course leading to a certificate proficiency. Tuition, $25 per year for residents of Georgia; $100 for no residents. The Georgia Chapter of the American Institute of Architects ha provided a loan-fund in this department for one or two students needi assistance.

Harvard University, Faculty of Architecture, Cambridge, Mas SCHOOL OF ARCHITECTURE. Professional training in architecture. (1) Ope to graduates of colleges, scientific schools and professional schools of good stan ing, leading to the degree of Master in Architecture, or Master in Architectu in Architectural Engineering. Length of period of study for men with professional preparation, commonly three years, depending on ability and pr vious training. (2) Open to competent special students, who must be ov twenty-one years of age, and must have had at least three years of offic experience; admitted to special course leading to certificate. Tuition $20 per year.

Howard University, Washington, D. C.

chool of Landscape-Architecture. (1) Professional training in landscape-hitecture, open to graduates of colleges and technical schools of good standing, ling to the degree of Master in Landscape-Architecture. (2) Special lents admitted to courses for which their training fits them. Tuition, $200. 'wo Traveling-Fellowships, the Julia Amory Appleton and the Robin-, are offered for competition in alternate years, each having an annual ıe of $1 100, tenable for two years, for travel and study in Europe under direction of the School of Architecture. The Charles Eliot Fellow-› in Landscape-Architecture (stipend $1 100) is offered for travel and ly in landscape-architecture, under the direction of the School of Landscape-hitecture. These fellowships are open for competition to graduates in ıitecture and in landscape-architecture, respectively.

esident Scholarships. Two Austin Scholarships in Architecture and in Landscape-Architecture, annual value, $350. The Cummings olarship in Landscape-Architecture, annual value, $350. One Eveleth olarship in Architecture, annual value, $250. Three Scholarships Special Students in Architecture, open to competition to properly lified draughtsmen, annual value, $200. Six University Scholarships open egular students in Architecture or Landscape-Architecture, annual value, o. Other scholarships available to candidates of special claims as to resi-ce, college, or descent.

nternational Correspondence Schools, Scranton, Pa. A corporation ıed to furnish instruction by correspondence and to hold examinations to blish proficiency. The architectural course is designed particularly to meet wants of those already engaged in the building trades or drafting-room. ıcludes sixty-one subjects covering the elements of building-construction, onry, carpentry, plumbing, etc., and the principles of design, drawing, ering, and specification-writing. The tuition includes text-books and uction, that is, criticisms on written lessons, sent to the schools, and also vers to questions on subjects connected with the course, that may be asked the students. Information regarding fees can be obtained on inquiry. ter courses are available for building-contractors, building-foremen, and special courses in structural engineering.

wa State College of Agriculture and Mechanic Arts, Ames, Iowa.

ansas State, Agricultural College of, Manhattan, Kan. Department rchitecture. Full four-year course in architecture, leading to a degree in elor of Science. Tuition free to residents of the state. Incidental fees unt to about $12 a semester.

lassachusetts Institute of Technology, Boston, Mass. Two four-year ses are offered in architecture, leading to the degree of Bachelor of Science: Course in general architecture; (2) Course in architectural engineering. ortunities are offered in each course for advanced professional work leading e degree in (1) of Master in Architecture and in (2) of Master of Science. ial students must be college-graduates, or twenty-one years of age, with not than two years of office-experience. In all cases they must demonstrate r fitness for the work of the department by personal conference with the l of the department, or his representative, and by the presentation of rs from former employers, together with drawings covering their experience ılly as possible. All special students must take in their first year of residence e Institute courses in descriptive geometry and mechanical drawing, unless e subjects have been passed at the September examinations for advanced ding, or excuse from one or both has been obtained on the basis of equivalent

work accomplished elsewhere. Tuition, $250 per year. An ANNUAL TRAVEL FELLOWSHIP amounting to $1 000 is given solely on the basis of disting merit, candidates being received from both regular and special students. prizes, varying from $10 to $200 each, are equally divided between the re and the special students. Certain funds are available for the assistance of qualified regular students for undergraduate and for post-graduate work.

McGill University, Montreal, Canada. DEPARTMENT OF ARCHITECTU (1) Full five-year course leading to the degree of Bachelor of Architec (2) Competent special students are admitted to take a partial course, but university certificate is granted for this work. Tuition, $150 per year.

North Dakota Agricultural College, Fargo, N. D. DEPARTMENT ENGINEERING. Draughtsmen's and builders' course of three years (six mo each). Full four-year course in architecture, leading to Bachelor of Scienc Architecture. Full four-year course in Architectural Engineering, lea to Bachelor of Science in Architectural Engineering. Tuition free. amounting to $35 per year.

Ohio Mechanics' Institute, Cincinnati, Ohio. DEPARTMENT OF ARC TECTURE. Technical high-school course preparatory to architecture, co ing four years. Two-year intensive course in architecture. Evening cl in architectural drawing and allied building-trade subjects. Graduates grammar-schools are trained in draughting and elementary architect subjects simultaneously with their high-school subjects. Graduates of hi schools are trained intensively in technical architectural work, including co giate mathematics and sciences, and receive a Certificate of Proficiency Architecture. Tuition, $75 per year.

Ohio State University, Columbus, Ohio. COURSE IN ARCHITECTU Two four-year courses, leading to the degrees of Bachelor of Architecture Bachelor of Architectural Engineering. Tuition free.

Oklahoma Agricultural and Mechanical College, Stillwater, Ok DEPARTMENTS OF ARCHITECTURE AND ARCHITECTURAL ENGINEERING. F year course in Architecture and Architectural Engineering, leading to a deg of Bachelor of Science. Two-year special course for draftsmen, leading to certificate of completion in this work. Tuition free. The registration-fee $2 a semester.

Pennsylvania State College, State College, Pa. Course in Archi tural Engineering. Full four-year course, leading to the degree of Bach of Science in Architectural Engineering. Tuition is free. Incidental amount to about $30 per semester, these fees including the college fees course in architectural design.

Pratt Institute, Brooklyn, N. Y. Course in Architecture. SCHOOL FINE AND APPLIED ARTS. (1) Two-year course in architectural design. Two-year course in architectural construction. (3) Full three-year course architectural design and architectural construction. The course in ar tectural design aims to give students a general training that will prepare th to pursue the profession of architecture as competent assistants in archite offices, and leads to positions of responsibility and independence. The co in architectural construction aims to fit the student for general draughting builders' offices, or for general detailing and construction-work in an archit office, and leads to the position of superintendent of construction-work. Tuiti $80 per year.

Princeton University, Princeton, N. J. SCHOOL OF ARCHITECTU

e courses in Architecture: (1) For students enrolled as candidates for egree of Bachelor of Arts on graduation and for the degree of Master of Arts in Architecture after two years of graduate work. (2) For students have not begun the study of architecture in the sophomore year, but wish to receive the degree of Bachelor of Arts on graduation and the degree aster of Fine Arts in Architecture after two years of graduate work. 'or students entering the School as candidates for the degree of Master ne Arts in Architecture without previous study in architecture. For the ge student, three years and a half are required for this course. Tuition, a year for students on full time, and $40 for those on part time. Annual $15. The GRADUATE FELLOWSHIP AND SCHOLARSHIPS of the University pen to members of the School. They are over fifty in number, and range pend from $150 to $1 000 per annum.

ce Institute, Houston, Tex. ARCHITECTURAL DEPARTMENT. Full year course leading to the degree of Bachelor of Science in Architecture. on free.

chester Athenaeum and Mechanics Institute, Rochester, N. Y. RTMENT OF APPLIED ARTS. Three-year courses in Architectural Drawing Design, and Architectural Construction, leading to Diplomas. There lso courses for properly prepared students who do not wish to take the ma-courses. Tuition for full courses, $90 per year; for part-time students, r term of twelve weeks for one session per week.

se Polytechnic Institute, Terre Haute, Ind. DEPARTMENT OF ARCHI-TRAL ENGINEERING. Full four-year course, designed to give a thorough ing in architectural engineering, together with systematic instruction in tectural design. Tuition and incidental fees, $110.

tch Traveling-Scholarship, Inc. (For particulars address the Secretary, eacon Street, Boston, Mass.) Candidates must be under thirty years of t the date of the beginning of the preliminary examinations. At that they must have been engaged in professional work during two years assachusetts in the employ of a practicing architect resident in Massa-tts, and will be required to pass preliminary examinations upon the follow-subjects: (1) History of architecture; (2) Freehand drawing from the (3) Construction, theory and practice; (4) An elementary knowledge e French language. Holders of a degree in Architecture from the Massa-tts Institute of Technology, Columbia University, University of Penn-nia, Cornell University, Harvard University, or University of Illinois be allowed to present such diploma which will be accepted in lieu of the inations in the preliminaries. Candidates who pass in these preliminary inations are admitted to a competition in design, the successful can-e in which is awarded the scholarship and receives annually, for two , $1 400, to be expended in foreign travel and study. The Boston Society chitects, through a committee, has complete charge of the examinations, supervises the work of the scholar. The Society of Architects awards um of $75 as a second prize.

racuse University, Syracause, N. Y., College of Fine Arts. DEPART-OF ARCHITECTURE. This school offers: Four-year courses in (1) Archi-re, (2) Architectural Design, (3) Architectural Engineering, all leading e degree of Bachelor of Architecture (B.Ar.); (4) Special two-year course rchitectural draughtsmen of two or more years' experience; (5) Graduate e in architecture; (6) Interior architectural design and decoration.

Tuition, $150 per year. Bulletins and full information available fr… Registrar.

Texas. Agricultural and Mechanical College of Texas, College S… Tex. DEPARTMENT OF ARCHITECTURE. Four-year course in archi… offering an option through the junior and senior years in architectural e… ing. Qualified special students admitted. Tuition free.

Tulane University of Louisiana, New Orleans, La. DEPARTME… ARCHITECTURE IN THE COLLEGE OF TECHNOLOGY. (1) Full four-year c… leading to a degree in architecture. (2) Special courses for students not… didates for a degree. Tuition, $100 per year. Special attention giv… subtropical conditions.

University of California, Berkeley, Cal. SCHOOL OF ARCHITEC… (1) Full four-year course leading to the degree of Bachelor of Arts. (2) … year-graduate course leading to the degree of Master of Arts. (3) Tw… graduate course leading to the degree of Graduate in Architecture. (4) Spec… elective courses for students not candidates for a degree. Tuition fr… residents of the state of California.

University of Illinois, Urbana, Ill. COURSES IN ARCHITECTU… ARCHITECTURAL ENGINEERING. (1) Full four-year course leading to… degree of Bachelor of Science in Architecture. (2) Full four-year co… leading to the degree of Bachelor of Science in Architectural Engine… Tuition is free. Incidental fee, $30 per year. PLYM TRAVELING-FELLOW… $1 000 for one year of travel abroad; awarded by competition to gradua… the Department of Architecture of the University of Illinois.

University of Kansas, Lawrence, Kan. DEPARTMENT OF ARCHITEC… AND ARCHITECTURAL ENGINEERING. Full four-year course in Architect… leading to the degree of Bachelor of Science in Architecture. Full four-… course in Architectural Engineering, leading to the degree of Bachelor of Sci… in Architectural Engineering. Four-year courses in each, based on one… in the College of Liberal Arts, leading to the degree of Bachelor of Sci… Tuition free. Fees amounting to $15 per year for residents of the state… $25 per year for non-residents.

University of Michigan, Ann Arbor, Mich. COLLEGE OF ARCHITEC… (1) A general four-year course leading to the degree of Bachelor of Scien… Architecture. (2) A four-year course in which architectural design is… phasized, leading to the same degree. (3) A four-year course in which th… a large proportion of engineering subjects, leading to the degree of Bach… of Science in Architectural Engineering. (4) Five-year courses leading to… degrees of Master of Science in Architecture and Master of Science in Arc… tectural Engineering. (5) A two-year course, leading to a Certificate, for sp… students (experienced draughtsmen or college-graduates). (6) Students m… earn the degree of Bachelor of Arts and the degree in Architecture in… five to six years. There are TWO SCHOLARSHIPS. Annual fees, $57 for stu… from Michigan and $87 for others.

University of Minnesota, Minneapolis, Minn. DEPARTMENT OF ARC… TECTURE. Full four-year course, leading to the degree of Bachelor of Sci… in Architecture. Fifth year, leading to the degree of Master of Scienc… Architecture. Special students of maturity and practical experience… admitted. Instruction is provided in Architectural Engineering. Tuition f… Incidental fee, $60 per year.

University of Nebraska, Lincoln, Neb. COLLEGE OF ENGINEER…

four-year course in architectural engineering, leading to Bachelor of Science rchitectural Engineering. Tuition free. Total fees for four years, $110.

niversity of Notre Dame, Notre Dame, Ind. DEPARTMENT OF ARCHI-URE. (1) Full four-year course in design leading to the degree of Bachelor cience in Architecture. (2) Full four-year course in architectural enging leading to the degree of Bachelor of Science in Architectural Engineering. Two-year special course leading to a Certificate of Proficiency. Tuition, per year; room $60 and upwards; board, $180 and upwards.

niversity of Oregon, Eugene, Ore. SCHOOL OF ARCHITECTURE AND ED ARTS. Two architectural options in design and structural work. Four-year course leading to the degree of Bachelor in Architecture. Five-year course leading to the degree of Master in Architecture. (3) Exon-courses in Portland, Ore., in design, etc. (4) Special courses for exnced draughtsmen. Tuition free for university-courses; $5 a term for nsion-courses.

niversity of Pennsylvania, Philadelphia, Pa. SCHOOL OF FINE ARTS, ARTMENT OF ARCHITECTURE. (1) Four-year course leading to the degree achelor of Architecture. (2) Graduate course of one year, with choice een major subjects, leading to the degree of Master of Architecture. Two-year special course leading to a professional certificate. (4) Six-year ngement of courses in liberal arts and architecture leading to the degrees . B. and also B. Arch. (5) Option in Architectural Engineering leading to degree of Bachelor of Architecture. Summer school providing instruction any architectural subjects of the regular session. The degree and certificate accepted by the American Institute of Architects in satisfaction of its ational requirements for membership and are credited by State Boards for sing of architects. Tuition $300 per year. Circular, including information ll courses in the School of Fine Arts, on application to the Dean of the School ine Arts, University of Pennsylvania, Philadelphia, Pa.

HE WOODMAN SCHOLARSHIP IN ARCHITECTURE of the University of Pennania, for one year of foreign travel and study, is open to graduates of this ol, they being also eligible to the general competition for the FELLOWSHIP HE AMERICAN ACADEMY IN ROME. The PARIS PRIZE OF THE BEAUX-ARTS ITUTE OF DESIGN is open to seniors and graduates and the STEWARDSON VELING-SCHOLARSHIP is available to students who are residents of Pennania. The MEDALS OF THE AMERICAN INSTITUTE OF ARCHITECTS and the ÉTÉ DES ARCHITECTES DIPLÔMÉS are conferred in this school as well as ER MEDALS AND PRIZES open to its students alone.

niversity of Santa Clara, Santa Clara, Cal.

niversity of Southern California, Los Angeles, Cal. Four-year general se in architecture, leading to the degree of B.S. in Architecture.

niversity of Texas, Austin, Tex. SCHOOL OF ARCHITECTURE. (1) Four- and five-year courses leading, respectively, to the degrees of Bachelor cience in Architecture, and Master of Science in Architecture. (2) Four- course leading to the degree of Bachelor of Science in Architectural Enering. Tuition free.

niversity of Toronto, Toronto, Canada. DEPARTMENT OF ARCHI-TURE. Full four-year course leading to the degree of Bachelor of Applied nce (B.A.Sc.) with an option of architectural engineering, replacing archiural design in the fourth year. The fees are, first year, $100; second year, o; third and fourth years, $120. The university is supported by the vince of Ontario.

University of Virginia. McIntire School of Fine Arts. Fo[ur-year] course in architecture, leading to the degree of Bachelor of Science in [Archi]tecture. Annual average of tuition and laboratory fees: For non-Virg[inians,] $180; for Virginians, $75.

University of Washington, Seattle, Wash. Course in Archite[cture]. Four-year course, leading to the degree of Bachelor of Architecture. Th[ere is] a fourth-year option in architectural engineering. Tuition, $20 per [year]. Entrance fee, $10; graduation fee, $5.

Washington, The State College of, Pullman, Wash. Departmen[t of] Architecture. (1) Full four-year course leading to the degree of Bach[elor] of Science in Architecture. (2) Two-year special course leading to a Certi[ficate] of Proficiency. (3) Special students, adequately prepared, are admitted t[o] classes. Tuition free.

Washington University, St. Louis, Mo. School of Architec[ture]. (1) Four-year courses in architecture and in architectural engineering le[ading] to the degrees of Bachelor of Architecture, and Bachelor of Science in A[rchi]tectural Engineering, respectively. (2) One-year course leading to the de[gree] of Master of Architecture. (3) Special two-year course with Certific[ate]. Tuition, $150 per year.

Wentworth Institute, Boston, Mass. Courses in architectural c[on]struction, carpentry and building, and twelve other technical trades or in[dus]tries. (1) Two-year course in architectural construction trains for posit[ions] of foremen, superintendents, detail-designers, etc. Tuition, $54 per year a[nd] $15 laboratory fee. (2) One-year course in carpentry and building pla[nned] for those wishing to enter the wood-working-trades and industries as advan[ced] apprentices or high-grade artisans. Tultion $30 per year and $15 laboratory f[ee].

Yale University, New Haven, Conn. Department of Architectu[re]. Regular course covers four years. Special degree, Bachelor of Fine Arts, [may] be competed for at end of course. Portions of the first-year's work, incl[uding] lectures on history of chief styles of architecture and principles of composi[tion] and practice in elementary design, may be taken as electives by juniors a[nd] seniors in the academic course. Alice Kimball English Scholarship, [sup]ported from fund of $11 000, for a year's travel abroad. William Wirt W[in]chester Scholarship, supported from fund of $20 000, for a year's t[ravel] abroad. Tuition, $180 per year.

ARCHITECTURAL SOCIETIES AND ORGANIZATIONS OF THE WORLD

1. United States

(1) THE AMERICAN INSTITUTE OF ARCHITECTS

The Octagon, Washington, D. C.

List of Chapters (1923) of the The American Institute of Archite[cts]

The year indicates the date of the chapter's organization

Alabama Chapter. 1916
Baltimore Chapter. 1870
Boston Chapter. 1870
Brooklyn Chapter. 1894
Buffalo Chapter. 1890
Central New York Chapter. 1887
Cincinnati Chapter. 1870
Cleveland Chapter. 1890
Colorado Chapter. 1892.
Columbus (Ohio) Chapter. 1913

ecticut Chapter. 1902
on Chapter. 1899
gia Chapter. 1906
is Chapter. 1869
Chapter. 1903
as City Chapter. 1890
ucky Chapter. 1908
siana Chapter. 1910
igan Chapter. 1887
esota Chapter. 1892
aska Chapter. 1919
Jersey Chapter. 1900
York Chapter. 1867
h Carolina Chapter. 1913
on Chapter. 1911

Philadelphia Chapter. 1869
Pittsburgh Chapter. 1891
Rhode Island Chapter. 1875
St. Louis Chapter. 1890
San Francisco Chapter. 1881
South Carolina Chapter. 1913
Southern California Chapter. 1894
Southern Pennsylvania Chapter. 1909
Tennessee Chapter. 1919
Texas Chapter. 1913
Toledo Chapter. 1914
Virginia Chapter. 1914
Washington (D. C.) Chapter. 1887
Washington State Chapter, 1894
Wisconsin Chapter. 1911

se chapters were organized since 1920: Arkansas, Central Illinois, Erie, Florida, a, Kansas State, Montana, St. Paul, Scranton-Wilkesbarre, South Georgia, and

: of State Associations of The American Institute of Architects

York State Society of Architects. 1919
State Association. 1915
sylvania State Association. 1909

(2) MISCELLANEOUS SOCIETIES *

ican Society of Landscape Architects
itects' Association of Indianapolis
itectural Club of Minneapolis
itectural League of Pacific Coast
itectural League of New York
itectural Society of the University of California
itectural Society of the University of Pennsylvania
iation of Collegiate Schools of Architecture
more Architectural Club
ingham Society of Architects
on Architectural Club
on Society of Architects
klyn Institute of Arts and Sciences
ago Architects' Business Association
ago Architectural Club
ago Association of Architects
nnati Architectural Club
eland Architectural Club
mbus Society of Architects
oit Architectural Club
th Architectural Club
neers' and Architects' Club of Louisville, Ky.
da Association of Architects
oyle Club of St. Paul
gia Architectural Association
napolis Architectural Club
as State Architects' Association

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anges are necessarily made in these lists from time to time.

Los Angeles Architectural Club
Massachusetts Institute of Technology Architectural Association
Minneapolis Architectural Club
Minneapolis Society of Architects
New Orleans Architectural Club
New York Society of Architects
Norfolk Society of Architects
North Carolina Architectural Association
Oakland Architects' Association
Oakland Architectural Club
Oklahoma State Association of Architects
Pittsburgh Architectural Club
Portland, Oregon, Architectural Club
Portland, Oregon, Association of Architects
St. Joseph, Missouri, Society of Architects
St. Louis Architectural Club
St. Paul Architectural Club
San Antonio Society of Architects
San Diego Architectural Association
San Francisco Architectural Club
Society of Architects of Akron, Ohio
Society of Architects of Columbia University
Society of Beaux-Arts Architects
Society of Naval Architects and Marine Engineers
South Bend Architectural Club
South Carolina Association of Architects
Southern States Engineering Society
Spokane Architectural Club
T Square Club of Philadelphia
Tacoma Society of Architects
Texas State Association of Architects
Utah Association of Architects
Washington, D. C., Architectural Club

2. Argentine Republic

Sociedad Central de Arquitectos. Buenos Aires

3. Austria

Austrian Society of Civil Engineers and Architects. Vienna
Architekten-Klub der Weiner Kunstlergenossenschaft. Vienna
Gesellschaft Österreichischer Architekten. Vienna
Weiner Bauhütte. Vienna
Towarzystwo Politechniczne we Lwowie. Leopol
Towarzystwo Techniscזne we Krakowie. Cracow

4. Belgium

Association des Architectes, de Liège. Liège
Société Centrale D'Architecture de Belgique. Brussels
Société Royale des Architectes D'Anvers. Antwerp
Kring Voor Bouwhunde D'Anvers. Antwerp
Chambre Syndicale des Architectes de Bruxelles. Brussels

ciation des Architectes de Bruxelles. Brussels
ité des Architectes de la Flandre Orientale. Ghent
ité des Architectes de la Flandre Orientale. Bruges

5. Bulgaria

ité des Ingénieurs et des Architectes Bulgares. Sofia

6. Canada

ARCHITECTURAL ASSOCIATION OF CANADA

ral Architectural Institute of Canada. Montreal
erta Association of Architects. Calgary and Edmonton, Alta.
hitects' Association of Victoria. Victoria, B. C.
ish Columbia Association of Architects.
gary Architectural Club
nitoba Association of Architects. Winnipeg, Man.
ario Association of Architects. Toronto
vince of Quebec Association of Architects. Montreal
gina Architectural Association. Regina, Sask.
katchewan Association of Architects. Regina, Sask.

7. Cuba

iety of Engineers and Architects of Havana. Havana

8. France

manent Committee of International Congresses of Architects. Paris
iété des Architectes Diplômés par le Gouvernement. Paris.
iété Nationale des Architectes de France. Paris.
iété Centrale des Architectes Français. Paris
ion Syndicale des Architectes Français. Paris
iété des Diplômés de l'École Spéciale d'Architecture. Paris
ociation Provenciale des Architectes Français. Versailles
iété Régionale des Architectes du Centre de la France. Bourges
iété Regionale des Architectes de Dauphiné et de la Savoie. Grenoble
iété des Architectes de l'Est de la France. Nancy
iété Régionale des Architectes du Limousin, de l'Angoulême et du Perigord. Guéret (Creuse)
iété Régionale des Architectes du Midi. Toulouse
iété Régionale des Architectes du Nord. Lille
iété Régionale des Architectes du Poitou et de la Saintonge. Parthenay
iété Régionale des Architectes du Puy-de-Dôme, du Cantal, de la Haute-Loire et de l'Allier. Clermont-Ferrand
iété Régionale des Architectes de Saône-et-Loire, de l'Ain et du Jura. Châlons-sur-Saône
ociation Régionale des Architectes du Sud-Est. Nice
iété des Architectes de l'Aisne. St. Quentin
iété des Architectes de l'Allier. Moulins
iété des Architectes de l'Anjou. Angers
iété des Architectes de l'Aube. Troyes
iété des Architectes de Blois. Blois
iété des Architectes de Bordeaux et du Sud-Ouest. Bordeaux
iété des Architectes des Bouches-du-Rhône. Marseilles

Société des Architectes du Doubs; Besançon
Société des Architectes de la Drôme et de l'Ardèche. Valence
Société des Architectes d'Eure-et-Loir. Chartres
Société Amicale et Syndicat des Architectes du Gard. Nîmes
Société des Architectes de la Haute-Marne. Châlons-sur-Marne
Société Académique d'Architecture de Lyon. Lyon
Société des Architectes de la Marne. Paul-Chandon
Société des Architectes de Nantes. Nantes
Société des Architectes de l'Oise. Compiègne
Société des Architectes d'Orléans. Orléans
Société des Architectes de Rennes. Rennes
Société des Architectes de la Seine Inférieure et de l'Eure. Rouen
Société des Architectes de Seine-et-Marne. Melun
Société des Architectes de Seine-et-Oise. Versailles
Société des Architectes de la Touraine. Tours
Société des Architectes de l'Yonne. Joigny
Association Amicale des Architectes. Paris
Réunion Amicale des Anciens Élèves de l'Atelier Questel-Pascal. Paris
Union Mutuelle des Architectes. Paris
Association Provinciale des Architectes Français. Bordeaux
Société des Architectes de la Côte-d'Or. Dijon
Société des Architectes du Nord-Ouest. Guingamp (Côtes-du-Nord)
Société des Architectes de la Loire. Saint-Étienne
Société des Architectes du Loiret. Orleans
Société des Architectes, Géomètres et Experts de la Lozère. Mende
Syndicat des Architectes du Rhône. Villeurbanne
Société des Architectes du Havre. Le Havre (Seine-Inférieure)
Union Architecturale de Lyon. Lyon
Association des Architectes Français. Marseilles
Syndicat des Architectes de Basse-Normandie. Caen
Société Historique de Compiègne. Compiègne
Société d'Assistance Confraternelle des Architectes Français. Versailles

9. Germany

Architekten Verein zu Berlin. Berlin. W.
Verbund. Deutscher Architekten und Ingenieur Verein. Berlin, S. W.
Württembergerischer Verein fur Baukunde. Stuttgart
Sächsischer Ingenieur und Architekten Verein. Dresden
Vereinigung Berliner Architekten. Berlin. W.
Architekten und Ingenieur Verein zu Hannover. Hannover
Architekten und Ingenieur Verein zu Osnabrück. Osnabrück
Architekten und Ingenieur Verein zu Hamburg. Hamburg
Architekten und Ingenieur Verein zu Cassel. Cassel
Architekten und Ingenieur Verein zu Lübeck. Lübeck
Schleswig-Holsteinischer, Architekten und Ingenieur Verein. Kiel
Baierischer Architekten und Ingenieur Verein. Munich
Architekten und Ingenieur Verein zu Breslau. Breslau
Badischer Architekten und Ingenieur Verein. Karlsruhe
Architekten und Ingenieur Verein zu Oldenburg. Oldenburg
Ostpreussischer Architekten und Ingenieur Verein. Königsberg
Frankfurter Architekten und Ingenieur Verein. Frankfurt-on-Main
Westpreussicher Architekten und Ingenieur Verein zu Danzig. Danzig
Architekten und Ingenieur Verein für Elsass Lothringen. Strassburg

elrheinischer Architekten und Ingenieur Verein. Darmstadt
dener Architekten Verein. Dresden
iitekten und Ingenieur Verein für Niederrhein und Westfalen. Cologne
in Leipziger Architekten. Leipzig
iitekten und Ingenieur Verein für das Herzogtum Braunschweig. Brunswick
iitekten und Ingenieur Verein zu Madgeburg. Magdeburg
iitekten und Ingenieur Verein zu Bremen. Bremen
iitekten und Ingenieur Verein zu Aachen. Aix-la-Chapelle
iitekten und Ingenieur Verein zu Metz. Metz
klenburgischer Architekten und Ingenieur Verein zu Schwerin, i.M. Schwerin
einigung Berliner Architekten. Berlin. W.
iitekten und Ingenieur Verein zu Düsseldorf. Düsseldorf
mberger Architekten und Ingenieur Verein. Bromberg
iitekten und Ingenieur Verein zu Münster, i.W. Münster
hitekten und Ingenieur Verein zu Potsdam. Potsdam
hitekten und Ingenieur Verein zu Stettin. Stettin
hitekten und Ingenieur Verein zu Posen. Posen
hitekten und Ingenieur Verein zu Erfurt. Erfurt
ein der Architekten und Bauingenieur zu Dortmund. Dortmund
einingung Schlesischer Architekten. Breslau
varzystwo Przyjaciol Nauk. Posen

10. Great Britain

yal Institute of British Architects. London, W.
rthern Architectural Association. Newcastle-upon-Tyne
ds and Yorkshire Architectural Society. Leeds
ffield Society of Architects and Surveyors. Sheffield
nchester Society of Architects. Manchester
erpool Architectural Society (Inc.). Liverpool
ttingham Architectural Association. Nottingham
mingham Architectural Association. Birmingham
cester and Leistershire Society of Architects. Leicester
stol Society of Architects. Bristol
rdiff, South Wales and Monmouth Architects' Society. Cardiff
von and Exeter Architectural Society. Exeter
isgow Institute of Architects. Dundee
ndee Institute of Architects. Dundee
erdeen Society of Architects. Aberdeen
inburgh Architectural Association. Edinburgh
rk and Yorkshire Architectural Society. York
yal Institute of Architects of Ireland (Inc.). Dublin
chitectural Association of Ireland. Dublin
stitute of Architects of New South Wales (Inc.). Sydney
yal Victorian Institute of Architects (Inc.). Melbourne
est Australian Institute of Architects (Inc.). Perth
ipe Institute of Architects. Cape Town, South Africa
ansvaal Institute of Architects. Johannesburg. Transvaal, South Africa
atal Institute of Architects. Durban. Natal, South Africa
ie Architectural Association. London, E.C.
ciety of Architects, London.

11. Greece

Hellenic Polytechnical Society. **Athens**

12. Holland

Society for the Propagation of Architecture. **Amsterdam**
Genootschap Architectura et Amicitia. **Amsterdam**
Bouwkunst en Vriendschap. **Rotterdam**

13. Hungary

Society of Engineers and of Architects. **Budapest**
Magyar Mernok-es Epitesz-Egylet. **Budapest**
Society of Private Architects. **Budapest**

14. Italy

Societa degli Ingegnerie e degli Architetti. **Rome**
Associazione Artistica fra i Cultori di Architettura. **Rome**
College des Ingenieurs et des Architectes de Gênes. **Gênes**
Collegio degli Ingegneri ed Architetti in Palermo. **Palermo**
Collegio Toscano degli Ingegneri ed Architetti in Firenze. **Florence**
Societa degli Ingegneri di Bologna. **Bologna**
Collegio degli Ingegneri ed Architetti di Milano. **Milan**
Collegio degli Ingegneri ed Architetti di Torino. **Turin**
Collegio degli Ingegneri ed Architetti di Messina.
Collegio degli Ingegneri ed Architetti Puglie. **Bari**
Collegio Veneto degli Ingegneri Venezia. **Venice**

15. Japan

Society of Architects. **Tokyo**

16. Norway

Société des Architectes et des Ingenieurs. **Christiania**

17. Portugal

Real Associo dos Architectos Civis e Archeologos Portuguezes. **Lisbon**
Sociedad dos Architectos Portuguezes. **Lisbon**

18. Russia

Société Impériale des Architectes Russes. **Petrograd**
Société des Architectes de Moscow. **Moscow**
Stowarzyszenie Technikow Kolo Architektow. **Varsovie**

19. Spain

Sociedad Centrale de Arquitectos de Madrid. **Madrid**
Associacion des Architectes de Cataluna. **Bajos**
Associacion des Architectes de Vizcaya. **Bilboa**
Associacion des Architectes de Navarra. **Pamplona**
Associacion de Arquitectos de Valencia. **Valencia**
Associacion de Arquitectos de Galicia. **Santiago (Coruna)**
Associacion de Arquitectos de Guipuzcoa. **San Sebastian**
Agrupacion Regional Central de Arquitectos de Castilla la Nueva. **Madrid**

ıpacion Regional de Arquitectos de Castilla la Vieja. Zamora
ıpacion Regional de Arquitectos de Norte. Bilboa
ıpacion Regional de Arquitectos de Catalana-Balear. Bajos
ıpacion Regional de Arquitectos de Andalucia. Cadiz
ıpacion Regional de Arquitectos de Galicia. Santiago (Coruna)
ıpacion Regional de Arquitectos de Cantabrico-Leonesa. Santander
ıpacion Regional de Arquitectos de Aragon. Teruel
ıpacion Regional de Arquitectos de Levante. Valencia
ıpacion Regional de Arquitectos de Canarias. Canaries
ıpacion Regional de Arquitectos de Occidente. Caceres

20. Sweden

été des Architectes et Ingenieurs. Stockholm
ska Teknologforenique. Stockholm

21. Switzerland

eizerischer Ingenieur und Architekten Verein. Bâle

22. Venezuela

edad de Arquitectura y Construccion du Venezuela. Caracas

GLOSSARY *

Technical Terms, Ancient and Modern, Used by Architects, Buil and Draughtsmen

Aaron's-Rod. An ornamental figure representing a rod with a serpent twi about it. It is sometimes confounded with the caduceus of Mercury. T distinction between the caduceus and the Aaron's-rod is that the former has tw serpents twined in opposite directions, while the latter has but one.

Abacus. The upper member of the capital of a column. It is sometim square and sometimes curved, forming on the plan segments of a circle called the arch of the abacus, and is commonly decorated with a rose or other ornament in the center, having the angles, called horns of the abacus, cut off in the direction of the radius or curve. In the Tuscan or Doric, it is a square tablet; in the Ionic, the edges are molded; in the Corinthian, its sides are concave and frequently enriched with carving. In Gothic pillars it has a great variety of forms.

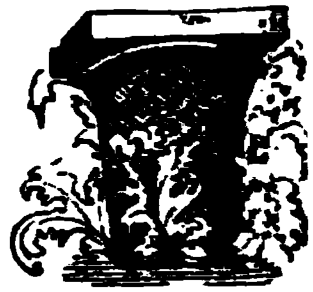
CORINTHIAN ABACUS

Abbey. A term for the church and other buildings used by conventual bodies presided over by an abbot or abbess, in contradistinction to cathedral, which is presided over by bishop; and priory, the head of which was a prior or prioress.

Abutment. That part of a pier from which the arch springs.

Abuttals. The boundings of a piece of land on other land, street, river, etc

Acanthus. A plant found in the south of Europe, representations of whose leaves are employed for decorating the Corinthian and Composite capitals. The leaves of the acanthus are used on the bell of the capital, and distinguish the two rich orders from the three others.


ACANTHUS

Acroteria. The small pedestals placed on the extremities and apex of a pediment. They are usually without bases or plinths, and were originally intended to receive statues.

Aile, Aisle. The wings; inward side porticos of a church; the inward latera corridors which enclose the choir, the presbytery, and the body of the church along its sides. Any one of the passages in a church or hall into which the pews or seats open.

Alcove. The original and strict meaning of this word, which is derived from the Spanish *alcoba*, is confined to that part of a bed-chamber in which the bed stands, separated from the other parts of the room by columns or pilasters. It is now commonly used to express any large recess in a room, generally separated by an arch.

Alipterion. In ancient Roman architecture, a room used by bathers fo anointing themselves.

* This Glossary was compiled by Mr. Kidder from various sources, and with th exception of some changes in typographical details to make it conform generally to th matter in the rest of the book it is left as published in the preceding editions.

lmonry. The place or chamber where alms were distributed to the poor in ırches, or other ecclesiastical buildings. At Bishopstone Church, Wiltshire, gland, it is a sort of covered porch attached to the south transept, but not nmunicating with the interior of the church. At Worcester Cathedral, Engd, the alms are said to have been distributed on stone tables, on each side, hin the great porch. In large monastic establishments, as at Westminster, eems to have been a separate building of some importance, either joining the e-house or near it, that the establishment might be disturbed as little as sible.

ltar. In ancient Roman architecture, a place on which offerings or sacris were made to the gods. In Protestant churches, the communion table is en designated as the Altar, and in Roman Catholic churches it is a square le placed at the east end of the church for the celebration of mass.

ltar of Incense. A small table covered with plates of gold on which was ced the smoking censer in the temple at Jerusalem.

ltar-piece. The entire decorations of an altar; a painting placed behind an ar.

ltar-screen. The back of the altar from which the canopy was suspended, l separating the choir from the lady chapel and presbytery. The Altar-screen s generally of stone, and composed of the richest tabernacle work of niches, als, and pedestals, supporting statues of the tutelary saints.

lto-rilievo. High relief. A sculpture, the figures of which project from the face on which they are carved.

mbo. A raised platform, a pulpit, a reading-desk, a marble pulpit — an ong enclosure in ancient churches, resembling in its uses and positions the dern choir.

mbry. A cupboard or closet, frequently found near the altar in ancient ırches to hold sacred utensils.

mbulatory. An alley — a gallery — a cloister.

mphiprostylos. A Grecian temple which has a columned portico on both ls.

mphitheater. A double theater, of an elliptical form on the plan, for the ibition of the ancient gladiatorial fights and other shows. Its arena or pit, in ich those exhibitions took place, was encompassed with seats rising above h other, and the exterior had the accommodation of porticos or arcades for public.

mphora. A Grecian vase with two handles, often seen on medals.

ncones. The consoles or ornaments cut on the key-stones of arches or on sides of door-cases. They are sometimes made use of to support busts or er figures.

ngle-bar. In joinery, an upright bar at the angles of polygonal windows; ıullion.

ngle-capital. In Greek architecture, those Ionic capitals placed on the flank umns of a portico, which have one of their volutes placed horizontally at an le of a hundred and thirty-five degrees with the plane of the frieze.

nnulated Columns. Columns clustered together by rings or bands; much d in English architecture.

nnular Vault. A vault rising from two parallel walls—the vault of a ridor. Same as *Barrel Vault.*

Annulet. A small square molding used to separate others. The fillet separates the flutings of columns is sometimes known by this term.

ANNULET

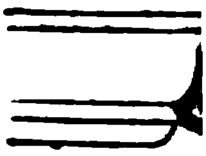
Anta, Antæ. A name given to a pilaster when attached to a wall. Vitruvius calls pilasters *parastatæ* when insulated. They are not usually diminished, and in all Greek examples their capitals are different from those of the columns they accompany.

Antechamber. An apartment preceded by a vestibule and from whic approached another room.

Antechapel. A small chapel forming the entrance to another. There examples at Merton College, Oxford, and at King's College, Cambridge, Engl besides several others. The antechapel to the lady-chapel in cathedral generally called the Presbytery.

Antechoir. The part under the rood loft, between the doors of the d and the outer entrance of the screen, forming a sort of lobby. It is also ca the Fore-choir.

Antefixa. In classical architecture (gargoyles, in Gothic architecture), ornaments of lions' and other heads below the eaves of a temple, through channels in which, usually by the mouth, the water is carried from the eaves. By some this term is applied to the upright ornaments above the eaves in ancient architecture, which hid the ends of the Harmi or joint tiles.



ANTEFIXA

Apophyge. The lowest part of the shaft of an Ionic or Corinthian colu or the highest member of its base if the column be considered as a whole. T Apophyge is the inverted cavetto or concave sweep, on the upper edge of wh the diminishing shaft rests.

Apron. A plain or molded piece of finish below the stool of a window, p on to cover the rough edge of the plastering.

Apse. The semicircular or polygonal termination to the chancel of a chur

Apteral. A temple without columns on the flanks or sides.

Aqueduct. An artificial canal for the conveyance of water, either above under ground. The Roman aqueducts are mostly of the former construction.

Arabesque. A building after the manner of the Arabs. Ornaments used the same people, in which no human or animal figures appear. Arabesque is sometimes improperly used to denote a species of ornaments composed of capricious fantastics and imaginary representations of animals and foliage so much employed by the Romans in the decorations of walls and ceilings.

Arabian Architecture. A style of architecture the rudiments of which appear to have been taken from surrounding nations, the Egyptians, Syrians, Chaldeans, and Persians. The best preserved specimens partake chiefly of the Græco-Roman, Byzantine, and Egyptian. It is supposed that they constructed many of their finest buildings from the ruins of ancient cities.

Aræostyle. That style of building in which the columns are distant from one another from four to five diameters. Strictly speaking, the term should be limited to intercolumniation of four diameters, which is only suited to the Tuscan order.

ARABESQ

Aræosystylos. That style of building in which four columns are used in the space of eight diameters and a half; the central

ercolumniation being three diameters and a half, and the others on each e being only half a diameter, by which arrangement coupled columns are roduced.

Arbores. Large bronze candelabra, in the shape of a tree, placed on the floor ancient churches, so as to appear growing out of it.

Arcade. A range of arches, supported either columns or on piers, and detached or attached the wall.

Arch. In building, a mechanical arrange-nt of building materials arranged in the form a curve, which preserves a given form when isting pressure, and enables them, supported piers or abutments, to carry weights and ist pressure.

ARCADE

Arch-buttress. Sometimes called a flying tress; an arch springing from a buttress or pier.

Architrave. That part of an entablature which rests upon the capital of a umn, and is beneath the frieze.

Architrave Cornice. An entablature consisting of an architrave and cor-, without the intervention of the frieze, sometimes introduced when incon-ient to give the entablature the usual height.

Architrave of a Door. The finished work surrounding the aperture; the er part of the lintel is called the traverse; and the sides, the jambs.

Archives. A repository or closet for the preservation of writings or records.

Archivolt. A collection of members forming the inner contour of an arch, a band or frame adorned with moldings running over the faces or the arch-nes, and bearing upon the imposts.

Area. The superficial contents of any figure; an open space or court within uilding; also, an uncovered space surrounding the foundation walls to give t to the basement.

Arena. The plain space in the middle of the amphitheater or other place of lic resort.

Arris. The meeting of two surfaces producing an angle.

Arsenal. A public storehouse for arms and ammunition.

Artificer, or **Artisan.** A person who works with his hands, and manufac-s any commodity in iron, brass, wood, etc.

Ashlar, or **Ashler.** A facing made of squared stones, or a facing made of slabs, used to cover walls of brick or rubble. *Coursed ashlar* is where the nes run in level courses all around the building, *random ashlar*, where the nes are of different heights, but level beds. Common freestones of small , as they come from the quarry, are also called ashlar.

Asphaltum. A kind of bituminous stone, principally found in the province Neufchatel. Mixed with stone, it forms an excellent cement, incorruptible ir and impenetrable by water.

Astragal. A small semicircular molding, sometimes plain and sometimes mented.

Asymptote. A straight line which continually approaches to a curve with-touching it.

Atlases, or **Atlantes.** Figures or half-figures of men, used instead of c umns or pilasters to support an entablature; called also Telamones.

ATLANTES

Atrium. A court in the interior division of Roman houses.

Attached Columns. Those which project three-fourths of their diameter from the wall.

Attic. A low story above an entablature, or above a cornice which limits the height of the main part of an elevation. Although the term is evidently derived from the Greek, we find nothing exactly answering to it in Greek architecture; but it is very common in both Roman and Italian practice. What are otherwise called tholobates in St. Peter's and St. Paul's Cathedrals are frequently termed attics.

Attic Order. A term used to denote the low pilasters employed in t decoration of an attic story.

Attributes. In painting and sculpture, symbols given to figures and stat to indicate their office and character.

Auditory. In ancient churches, that part of the church where the peo usually stood to be instructed in the Gospel, now called the nave.

Aula. A court or hall in ancient Roman houses.

Aviary. A large apartment for breeding birds.

Axis. The spindle or center of any rotative motion. In a sphere, an im inary line through the center.

Back-choir. A place behind the altar in the principal choir, in which th is, or was, a small altar standing back to back with the former.

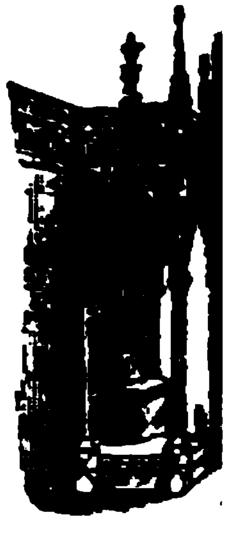
Backing of a Rafter or Rib. The forming of an upper or outer surfa that it may range with the edges of the ribs or rafters on either side.

Backing of a Wall. The rough inner face of a wall; earth deposited beh a retaining wall, etc.

Back of a Window. That piece of wainscoting which is between the bott of the sash frame and the floor.

Balcony. A projection from the face of a wall, supported by columns or c soles, and usually surrounded by a balustrade.

Baldachin. A building in the form of a canopy, supported with columns, a serving as a crown or covering to an altar.



BALDACHI

Baluster. A small pillar or column, supporting a rail, of various forms, used in balustrades.

Baluster Shaft. The shaft dividing a window in Saxon architecture. At St. Albans are some of these shafts, evidently out of the old Saxon church, which have been fixed up with Norman capitals.

Balustrade. A series of balusters connected by a rail.

Band. A sort of flat frieze or fascia running horizontally round a tower or other parts of a building, particularly the base tables in perpendicular work, commonly used with the long shafts characteristic of the thirteenth century. It generally has a bold, projecting molding above

ıd below, and is carved sometimes with foliages, but in general with cusped rcles, or quatrefoils, in which frequently are shields of arms.

Band of a Column. A series of annulets and hollows going round the middle the shafts of columns, and sometimes of the entire pier. They are often beau-fully carved with foliages, etc., as at Amiens. In several cathedrals there are ıgs of bronze apparently covering the junction of the frusta of the columns. : Worcester and Westminster they appear to have been gilt; they are there ore properly called Shaft-rings.

Baptistery. A separate building to contain the font, for the rite of baptism. ıey are frequent on the Continent; that at Rome, near St. John Lateran, and ose at Florence, Pisa, Pavia, etc., are all well-known examples. The only ex-ıples in England are at Cranbrook and Canterbury; the latter, however, is pposed to have been originally part of the treasury.

Barbican. An outwork for the defence of a gate or drawbridge; also, a sort pent-house or construction of timber to shelter warders or sentries from arrows other missiles.

Barge Board. See *Verge Board.*

Bartizan.. A small turret, corbeled out at the angle of a wall or tower, to pro-ct a warder and enable him to see around him. ıey generally are furnished with oylets or arrow-ts.

BARTIZAN

Basement. The lower part of a building, usu-y in part below the grade of the lot or street.

Base Moldings. The moldings immediately ove the plinth of a wall, pillar, or pedestal.

Base of a Column. That part which is between e shaft and the pedestal, or, if there be no pedes-l, between the shaft and the plinth. The Grecian Doric had no base, and the ıscan has only a single torus, or a plinth.

Basilica. A term given by the Greeks and Romans to the public buildings voted to judicial purposes.

Bas-relief. See *Basso-rilievo.*

Basse-cour. A court separated from the principal one, and destined for ıbles, etc.

Basso-rilievo, or **Bas-relief.** The representations of figures projected from background without being detached from it. It is divided into three parts: to-rilievo, when the figure projects more than one-half; Mezzo-rilievo, that in ıich the figure projects one-half; and Basso-rilievo, when the projection of the ıre is less than one-half, as in coins.

Bat. A part of a brick.

Batten. Small scantlings, or small strips of boards, used for various purposes. ıall strips put over the joints of sheathing to keep out the weather.

Batten-door. A door made of sheathing, secured by strips of board, put ssways, and nailed with clinched nails.

Batter. A term used by bricklayers, carpenters, etc., to signify a wall, piece timber, or other material, which does not stand upright, but inclines from you ıen you stand before it; but when, on the contrary, it leans toward you, it is d to overhang.

Battlement. A parapet with a series of notches in it, from which arrows may be shot, or other instruments of defence hurled on besiegers. The raised portions are called merlons; and the notches, embrasures or crenelles. The former were intended to cover the soldier while discharging his weapon through the latter. Their use is of great antiquity; they are found in the sculptures of Nineveh, in the tombs of Egypt, and on the famous François vase, where there is a delineation of the siege of Troy. In ecclesiastical architecture the early battlements have small shallow embrasures at some distance apart. In the Decorated period they are closer together, and deeper, and the moldings on the top of the merlon and bottom of the embrasure are richer. During this period, and the early part of the Perpendicular, the sides or cheeks of the embrasures are perfectly square and plain. In later times the moldings were continued round the sides, as well as at top and bottom, mitring at the angles, as over the doorway of Magdalen College, Oxford, England. The battlements of the Decorated and later periods are often richly ornamented by paneling, as in the last example. In castellated work the merlons are often pierced by narrow arrow-slits. (See *Oylet.*) In South Italy some battlements are found strongly resembling those of old Rome and Pompeii; in the Continental ecclesiastical architecture, the parapets are very rarely embattled.

BATTLEMENT

Bay. Any division or compartment of an arcade, roof, etc. Thus each space from pillar to pillar, in a cathedral, is called a bay, or severy.

Bay Window. Any window projecting outward from the wall of a building, either square or polygonal on plan, and commencing from the ground. If they are carried on projecting corbels, they are called Oriel windows. Their use seems to have been confined to the later periods. In the Tudor and Elizabethan style they are often semicircular in plan, in which case some think it more correct to call them Bow Windows.

Bazaar. A kind of Eastern mart, of Arabic origin.

Bead. A circular molding. When several are joined, it is called Reeding; when flush with the surface, it is called Quirk-bead; and when raised, Cock-bead.

Beam. A piece of timber, iron, stone, or other material, placed horizontally, or nearly so, to support a load over an opening, or from post to post.

Bearing. The portion of a beam, truss, etc., that rests on the supports.

Bearing Wall, or **Partition.** A wall which supports the floors and roofs in a building.

Beaufet, or **Buffet.** A small cupboard, or cabinet, to contain china. It may either be built into a wall, or be a separate piece of furniture.

Bed. In bricklaying and masonry, the horizontal surfaces on which the stones or bricks of walls lie in courses.

Bed of a Slate. The lower side.

Bed Moldings. Those moldings in all the orders between the corona and frieze.

Belfry. Properly speaking, a detached tower or campanile containing bells, as at Evesham, England, but more generally applied to the ringing-room or loft of the tower of a church. See *Tower*.

Bell-cot, Bell-gable, or **Bell-turret.** The place where one or more bells are ıung in chapels, or small churches which have no towers. Bell-cots are some- :imes double, as at Northborough and Coxwell, England; a very common form n France and Switzerland admits of three bells. In these countries, also, they ıre frequently of wood, and attached to the ridge. Those which stand on the gable, dividing the nave from the chancel, are generally called Sanctus Bells. A /ery curious and, it is believed, unique example at Cleves Abbey, England, juts ›ut from the wall. In later times bell-turrets were much ornamented; these are ›ften called Flèches.

Bell of a Capital. In Gothic work, immediately above the necking is a deep, ıollow curve; this is called the bell of a capital. It is often enriched with foli- ıges. It is also applied to the body of the Corinthian and Composite capitals.

Belt. A course of stones or brick projecting from a brick or stone wall, gen- ›rally placed in a line with the sills of the windows; it is either molded, fluted, ›lane, or enriched with patras at regular intervals. Sometimes called Stone ›tring.

Belvedere, or **Look-out.** A turret or lantern raised above the roof of an ›bservatory for the purpose of enjoying a fine prospect.

Bema. The semicircular recess, or hexedra, in the basilica, where the judges at, and where in after-times the altar was placed. It generally is roofed with a ıalf-dome or concha. The seats of the priests were against the wall, looking into he body of the church, that of the bishop being in the center. The bema ; generally ascended by steps, and railed off by cancelli.

Bench Table. The stone seat which runs round the walls of large churches, ınd sometimes round the piers; it very generally is placed in the porches.

Bevel. An instrument for taking angles. One side of a solid body is said to e beveled with respect to another, when the angle contained between those two ides is greater or less than a right angle.

Bezantee. A name given to an ornamental molding much used in the Nor- ıan period, resembling bezants, coins struck in Byzantium.

Billet. A species of ornamented molding much used in Norman, and some- imes in Early English work, like short pieces of stick cut off and arranged alter- ately.

Blocking, or **Blocking-course.** In masonry, a course of stones placed on he top of a cornice crowning the walls.

Bond. In bricklaying and masonry, that connection between bricks or stones ›rmed by lapping them upon one another in carrying up the work, so as to form n inseparable mass of building, by preventing the vertical joints falling over ach other. In brickwork there are several kinds of bond. In common brick alls in every sixth or seventh course the bricks are laid crossways of the wall, alled Headers. In face work, the back of the face brick is clipped so as to get a diagonal course of headers behind. In Old English bond, every alternate ›urse is a header course. In Flemish bond, a header and stretcher alternate each course.

Bond-stones. Stones running through the thickness of the wall at right ıgles to its face, in order to bind it together.

Bond-timbers. Timbers placed in a horizontal direction in the walls of a ick building in tiers, and to which the battens, laths, etc., are secured. In rub- e work, walls are better plugged for this purpose.

Border. Useful ornamental pieces around the edge of anything.

Boss. An ornament, generally carved, forming the key-stone at the intersetion of the ribs of a groined vault. Early Norman vaults have no bosses. The carving is generally foliage, and resembles that of the period in capitals, etc. Sometimes they have human heads, as at Notre Dame at Paris, and sometimes grotesque figures. In Later Gothic vaulting there are bosses at every intersection.

Boutell. The mediæval term for a round molding, or torus. When it follows a curve, as round a bench end, it is called a Roving Boutell.

Bow. Any projecting part of a building in the form of an arc of a circle. A bow, however, is sometimes polygonal.

Bow Window. A window placed in the bow of a building.

Brace. In carpentry, an inclined piece of timber, used in trussed partitions or in framed roofs, in order to form a triangle, and thereby stiffen the framing. When a brace is used by way of support to a rafter, it is called a strut. Braces in partitions and span-roofs are, or always should be, disposed in pairs, and introduced in opposite directions.

Brace Mold. [{] Two ressaunts or ogees united together like a brace in printing, sometimes with a small bead between them.

Bracket. A projecting ornament carrying a cornice. Those which support vaulting shafts or cross springers of a roof are more generally called Corbels.

Break. Any projection from the general surface of a building.

Breaking Joint. The arrangement of stones or bricks so as not to allow two joints to come immediately over each other. See *Bond*.

Breast of a Window. The masonry forming the back of the recess and the parapet under the window-sill.

CROSS-BRIDGING

Bressummer. A lintel, beam, or iron tie, intended to carry an external wall and itself supported by piers or posts; used principally over shop windows. This term is now seldom used, the word *beam*, or *girder*, taking its place.

Bridging. A method of stiffening floor joist and partition studs, by cutting pieces in between. Cross bridging of floor joist is illustrated in cut.

Bulwark. In ancient fortification, nearly the same as Bastion in modern.

Burse, or **Bourse.** A public edifice for the assembly of merchant traders; an exchange.

Bust. In sculpture, that portion of the human figure which comprises the head, neck, and shoulders.

BUTTRESS

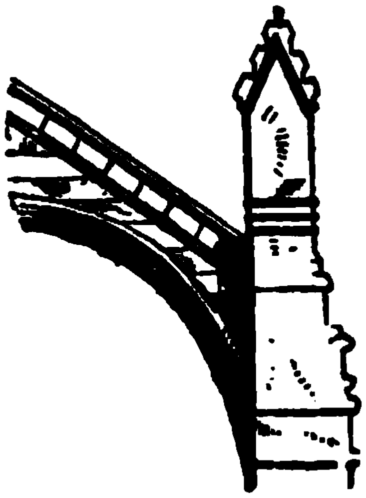
Buttery. A store-room for provisions.

Butt-joint. Where the ends of two pieces of timber or molding butt together.

Buttress. Masonry projecting from a wall, and intended to strengthen the same against the thrust of a roof or vault. Buttresses are no doubt derived from the classic pilasters which serve to strengthen walls where there is a pressure of a girder or roof-timber. In very early work they have little projection, and, in fact, are "strippilasters." In Norman work they are wider, with very little projection, and generally stop under a cornice or corbel table. Early English buttresses project considerably, sometimes with deep sloping weatherings in several

stages, and sometimes with gabled heads. Sometimes they are chamfered, and sometimes the angles have jamb shafts. At Wells and Salisbury, England, they are richly ornamented with canopies and statues. In the Decorated period they became richly paneled in stages, and often finish with niches and statues and elegantly carved and crocketed gablets, as at York, England. In the Perpendicular period the weatherings became waved, and they frequently terminate with niches and pinnacles.

Buttress, Flying. A detached buttress or pier of masonry at some distance from a wall, and connected therewith by an arch or portion of an arch, so as to discharge the thrust of a roof or vault on some strong point.



FLYING BUTTRESS

Buttress Shafts. Slender columns at the angle of buttresses, chiefly used in the Early English period.

Byzantine Architecture. A style developed in the Byzantine Empire. The capitals of the pillars are of endless variety and full of invention; some are founded on the Greek Corinthian, some resemble the Norman and the Lombard style, and are so varied that no two sides of the same capital are alike. They are comprised under the style Romanesque, which comprehends the round-arch style. Byzantine architecture reached its height in the Church of St. Sophia at Constantinople.

Cabinet. A highly ornamented kind of buffet or chest of drawers set apart for the preservation of things of value.

Cabling. The flutes of columns are said to be cabled when they are partly occupied by solid convex masses, or appear to be refilled with cylinders after they had been formed.

Caduceus. Mercury's rod, a wand entwined by two serpents and surmounted by two wings. The rod represents power; the serpents, wisdom; and the wings, diligence and activity.

CADUCEUS

Caisson. A panel sunk below the surface in flat or vaulted ceilings. See *Cassoon*

Caisson. In bridge building, a chest or vessel in which the piers of a bridge are built, gradually sinking as the work advances till its bottom comes in contact with the bed of the river, and then the sides are disengaged, being so constructed as to allow of their being thus detached without injury to its floor or bottom.

Caliber, or **Caliper.** The diameter of any round body; the width of the mouth of a piece of ordnance.

Camber. In carpentry, the convexity of a beam upon the surface, in order to prevent its becoming concave by its own weight, or by the burden it may have to sustain.

Campanile. A name given in Italy to the bell-tower of a town-hall or church. In that country this is almost always detached from the latter.

Candelabrum. Stand or support on which the ancients placed their lamps. Candelabra were made in a variety of shapes and with much taste and elegance. The term is also used to denote a tall ornamental candlestick with several arms, or a bracket with arms for candles.

Canopy. The upper part or cover of a niche, or the projection or ornament over an altar, seat, or tomb. The word is supposed to be derived from cono-

peum, the gauze covering over a bed to keep off the gnats; a mosquito curtain. Early English canopies are generally simple, with trefoiled or cinque-foiled heads, but in the later styles they are very rich, and divided into compartments with pendants, knots, pinnacles, etc. The triangular arrangement over an Early English and Decorated doorway is often called a canopy. The triangular canopies in the North of Italy are peculiar. Those in England are generally part of the arrangement of the arch moldings of the door, and form, as it were, the hood-molds to them, as at York. The former are above and independent of the door moldings, and frequently support an arch with a tympanum, above which is a triangular canopy, as in the Duomo at Florence. Sometimes the canopy and arch project from the wall, and are carried on small jamb shafts, as at San Pietro Martiro at Verona. Canopies are often used over windows, as at York Minster over the great west window, and lower tiers in the towers. These are triangular, while the upper windows in the towers have ogee canopies.

Capital. The upper part of a column, pilaster, pier, etc. Capitals have been used in every style down to the present time. That mostly used by the Egyptians was bell-shaped, with or without ornaments. The Persians used the double-headed bell, forming a kind of bracket capital. The Assyrians apparently made use of the Ionic and Corinthian, which were developed by the Greeks, Romans and Italians into their present well-known forms. The Doric was apparently an invention or adaptation by the Greeks, and was altered by the Romans and Italians. But in all these examples, both ancient and modern, the capitals of an order are all of the same form throughout the same building, so that if one be seen the form of all the others is known. The Romanesque architects altered all this, and in the carving of their capitals often introduced such figures and emblems as helped to tell the story of their building. Another form was introduced by them in the curtain capital, rude at first, but afterward highly decorated. It evidently took its origin from the cutting off of the lower angles of a square block, and then rounding them off. The process may be distinctly seen, in its several stages, in Mayence Cathedral. But this form of capital was more fully developed by the Normans, with whom it became a marked feature. In the early English capitals a peculiar flower of three or more lobes was used spreading from the necking upward in most graceful forms. In Decorated and Perpendicular styles this was abandoned in favor of more realistic forms of crumpled leaves, enclosing the bell like a wreath. In each style bold abacus moldings were always used, whether with or without foliage.

Caravansary. A huge, square building, or inn, in the East, for the reception of travelers and lodging of caravans.

Carriage. The timber or iron joist which supports the steps of a wooden stair.

CARYATID

Carton, or Cartoon. A design made on strong paper, to be transferred on the fresh plaster wall to be afterward painted in fresco; also, a colored design for working in mosaic tapestry.

Cartouche. An ornament which like an escutcheon, a shield or an oval or oblong panel has the central part plain, and usually slightly convex, to receive an inscription, armorial bearings, or an ornamental or significant piece of painting or sculpture. Frequently used in French Renaissance and Modern Architecture.

Caryatides. Human female figures used as piers, columns, or supports. *Caryatic* is applied to the human figure generally, when used in the manner of caryatides.

Cased. Covered with other materials, generally of a better quality.

Casement. A glass frame which is made to open by turning on hinges fixed to its vertical edges.

Cassoon, or Caisson. A deep panel or coffer in a soffit or ceiling. This rm is sometimes written in the French form, *caisson;* sometimes derived irectly from the Italian *cassone,* the augmentative of cassa, a chest or coffer.

Cast. A term used in sculpture for the impression of any figure taken in laster of Paris, wax, or other substances.

Catacombs. Subterranean places for burying the dead. Those of Egypt, nd near Rome, are believed to be the most important.

Catafalco. An ornamental scaffold used in funeral solemnities.

Cathedral. The principal church, where the bishop has his seat as diocesan.

Cauliculus. The inner scroll of the Corinthian capital. It is not uncommon, owever, to apply this term to the larger scrolls or volutes also.

Causeway. A raised or paved way.

Cavetto. A concave ornamental molding, opposed in effect to the ovolo — ne quadrant of a circle.

Ceiling. That covering of a room which hides the joists of the floor above, the rafters of the roof Most European churches either have open roofs, or re groined in stone. At Peterborough and St. Albans, England, there are very d flat ceilings of boards curiously painted. In later times the boarded ceilings, nd, in fact, some of those of plaster, have molded ribs, locked with bosses at ne intersection, and are sometimes elaborately carved. In many English hurches there are ceilings formed of oak ribs, filled in at the spandrels ith narrow, thin pieces of board, in exact imitation of stone groining. In ne Elizabethan and subsequent periods the ceilings are enriched with ost elaborate ornaments in stucco. Matched and beaded boards, planed nd smoothed, used for wainscoting. In the New England States it is called heathing.

Cenotaph. An honorary tomb or monument, distinguished from monuments being empty, the individual it is to memorialize having received interment sewhere.

Centaur. A poetical imaginary being of heathen mythology, half-man and alf-horse.

Centring. In building, the frames on which an arch is turned.

Chamfer, Champfer, or Chaumfer. When the edge or arris of any work is ut off at an angle of 45° in a small degree, it is said to be chamfered; if a large scale, it is said to be a canted corner. The chamfer is much used in ediæval work, and is sometimes plain, sometimes hollowed out, and sometimes olded.

Chamfer Stop. Chamfers sometimes simply run into the arris by a plane ce; more commonly they are first stopped by some ornament, as by a bead; ey are sometimes terminated by trefoils, or cinque-foils, double or single, and general form very pleasing features in mediæval architecture.

Chancel. A place separated from the rest of a church by a screen. The word now generally used to signify the portion of an Episcopal or Catholic church ontaining the altar and communion table.

Chantry. A small chapel, generally built out from a church. They generally ontain a founder's tomb, and are often endowed places where masses might

be said for his soul. The officiator, or mass priest, being often unconnected with the parochial clergy. The chantry has generally an entrance from the outside.

Chapel. A small, detached building used as a substitute for a church in a large parish; an apartment in any large building, a palace, a nobleman's house, a hospital or prison, used for public worship; or an attached building running out of and forming part of a large church, generally dedicated to different saints, each having its own altar, piscina, etc., and screened off from the body of the building.

Chapter House. The chamber in which the chapter or heads of the monastic bodies assembled to transact business. They are of various forms; some are oblong apartments, some octagonal, and some circular.

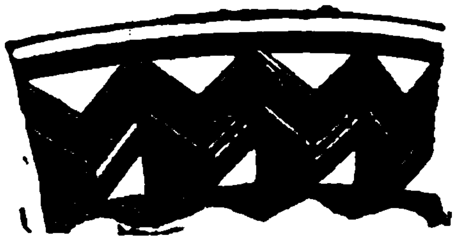
Chaptrel. In Gothic architecture, the capital of a pier or column which receives an arch.

CHAPTREL

Charnel House. A place for depositing the bones which might be thrown up in digging graves. Sometimes it was a portion of the crypt; sometimes it was a separate building in the church-yard; sometimes chantry chapels were attached to these buildings. M. Viollet-le-Duc has given two very curious examples of *ossuaires* — one from Fleurance, the other from Faouet.

Cherub — Gothic. A representation of an infant's head joined to two wings, used in the churches on key-stones of arches and corbels.

Chevron — Gothic. An ornament turning this and that way, like a zigzag, or letter Z.



CHEVRON

Chiaro-oscuro. The effects of light and shade in a picture.

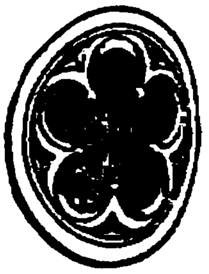
Choir. That part of a church or monastery where the breviary service, or "horæ," is chanted.

Church. A building for the performance of public worship. The first churches were built on the plan of the ancient basilicæ, and afterward on the plan of a cross: a church is said to be in Greek cross when the length of the transverse is equal to that of the nave; in Latin cross, when the nave is longer than the transverse part; in rotundo, when it is a perfect circle; simple, when it has only a nave and choir; with aisles, when it has a row of porticos in form of vaulted galleries, with chapels in its circumference.

Ciborium. A tabernacle or vaulted canopy supported on shafts standing over the high altar.

Cincture. A ring, list, or fillet at the top and bottom of a column, serving to divide the shaft of the column from its capital and base.

Cinque-foil. A sinking or perforation, like a flower, of five points or leaves, as a quatre-foil is of four. The points are sometimes in a circle, and sometimes form the cusping of a head.



CINQUE-FOIL

Civic Crown. A garland of oak-leaves and acorns, given as honorary distinction among the Romans to such as had preserved the life of a fellow-citizen.

Clere-story, Clear-story. When the middle of the nave of a church rises bove the aisles and is pierced with indows, the upper story is thus illed. Sometimes these windows e very small, being mere quatreils, or spherical triangles. In large uildings, however, they are importunt objects both for beauty and tility. The window of the clerecories of Norman work, even in large hurches, are of less importance than 1 the later styles. In Early English ney became larger; and in the Decoated they are more important still, eing lengthened as the triforium iminishes. In Perpendicular work he latter often disappears altogether, nd in many later churches the clerestories are close ranges of windows. 'he word *clere-story* is also used to enote a similar method of lighting ther buildings besides churches, especially factories, depots, sheds, etc.

Bath Abbey

FLYING BUTTRESS AND CLERE-STORY

A, buttress with pinnacle; B, flying buttress supporting clere-story; C, vaulted roof of aisle; D D, pier dividing nave from aisle; E, vaulted roof of nave.

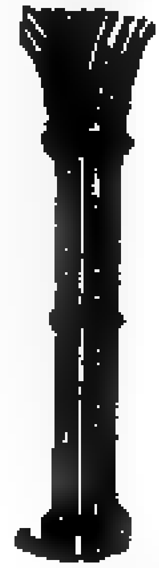
Cloister. An enclosed square, like he atrium of a Roman house, with a alk or ambulatory around, sheltered y a roof, generally groined, and by racery windows, which were more r less glazed.

Close. The precinct of a cathedral r abbey. Sometimes the walls are raceable, but now generally the oundary is only known by tradion.

Close String, or **Box String.** A ıethod of finishing the outer edge of airs, by building up a sort of curb ring on which the balusters set, nd the treads and risers stop against it.

Clustered. In architecture, the coalition of several members 'hich penetrate each other.

Clustered Column. Several slender pillars attached to each ther so as to form one. The term is used in Roman architecture denote two or four columns which appear to intersect each other the angle of a building to answer at each return.



CLUSTERED COLUMN

Coat. A thickness or covering of paint, plaster, or other work, one at one time. The first coat of plastering is called the scratch at, the second coat (when there are three coats) is called the brown at, and the last coat is variously known as the slipped coat, imcoat, or white coat. It varies in composition in different calities.

Coffer. A deep panel in a ceiling.

Coffer Dam. A frame used in the building of a bridge in deep water similar to a caisson.

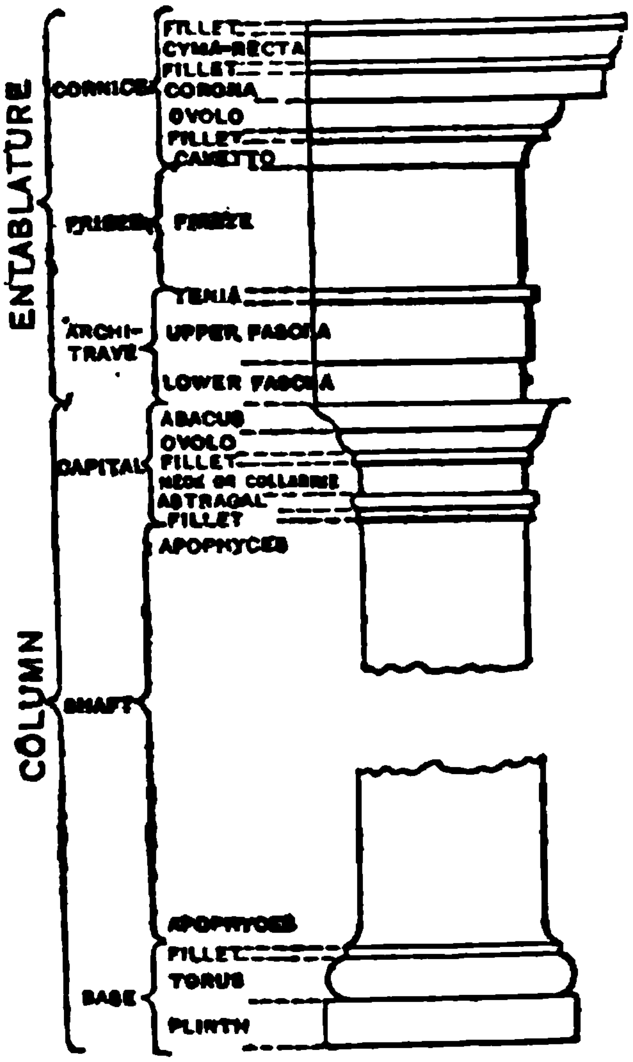
Collar Beam. A beam above the lower ends of the rafters, and spiked to them.

Colonnade. A row of columns. The colonnade is termed, according to the number of columns which support the entablature: Tetrastyle, when there are four; hexastyle, when six; octostyle, when eight, etc. When in front of a building they are termed porticos; when surrounding a building, peristyle; and when double or more, polystyle.

Colosseum, or Coliseum. The immense amphitheater built at Rome by Flavius Vespasian, A D. 72, after his return from his victories over the Jews. It would contain ninety thousand persons sitting, and twenty thousand more standing. The name is now employed to denote an unusually large audience building, generally of a temporary nature.

Colossus. The name of a brazen statue which was erected at the entrance of the harbor at Rhodes, one hundred and five feet in height. Vessels could sail between its legs.

Column. A round pillar. The parts are the base, on which it rests; its body, called the shaft; and the head, called the capital. The capital finishes with a horizontal table, called the abacus, and the base commonly stands on another, called the plinth. Columns may be either insulated or attached. They are said to be attached or engaged when they form part of a wall, projecting one-half or more, but not the whole, of their substance.



SECTION OF COLUMN AND ENTABLATURE

(Divided according to the Tuscan Order)

Common. A line, angle, surface, etc., which belongs equally to several objects. Common centring is a centring without trusses, having a tie beam at bottom. Common joists are the beams in naked flooring to which the joists are fixed. Common rafters in a roof are those to which the laths are attached.

Composite Arch. Is the pointed or lancet arch.

Composite Order. The most elaborate of the orders of classical architecture.

Compound Arch. A usual form of medieval arch, which may be resolved into a number of concentric archways, successively placed within and behind each other.

Conduit. A long narrow passage between two walls or underground for secret communication between different apartments; also, a canal or pipe for the conveyance of water.

Confessional. The seat where a priest or confessor sits to hear confessions

Conge. Another name for the echinus or quarter round.

Conservatory. A building for the protection and rearing of tender plants, ften attached to a house as an apartment. Also, a public place of instruction, esigned to preserve and perfect the knowledge of some branch of learning or he fine arts; as, a *conservatory* of music.

Consistory. The judicial hall of the College of Cardinals at Rome.

Consol, or Console. A bracket or truss, generally with scrolls or volutes at he two ends, of unequal size and contrasted, but onnected by a flowing line from the back of the pper one to the inner convolving face of the lower.

CONSOLES

Coping. The capping or covering of a wall. This of stone, weathered to throw off the wet. In Noran times, as far as can be judged from the little there left, it was generally plain and flat, and projected ver the wall with a floating to form a drip. Afterard it assumed a torus or bowtell at the top, and beame deeper, and in the Decorated period there were enerally several sets-off. The copings in the Perpendicular period assumed omething of the wavy section of the buttress caps, and mitred round the sides f the embrasure, as well as the top and bottom.

Corbel. The name, in medieval architecture, for a piece of stone jutting out f a wall to carry any superincumbent weight. A piece of timber projecting in he same way was called a tassel or a bragger. Thus, the carved ornaments from hich the vaulting shafts spring at Lincoln are corbels. Norman corbels are enerally plain. In the Early English period they are sometimes elaborately arved. They sometimes end with a point, apparently growing into the wall, r forming a knot, and often are supported by angles and other figures. In the ater periods the foliage or ornaments resemble those in the capitals. In modern rchitecture, a short piece of stone or wood projecting from a wall to form a upport, generally ornamented.

Corbel Out. To build out one or more courses of brick or stone from the face f a wall, to form a support for timbers.

Corbel Table. A projecting cornice or parapet, supported by a range of orbels a short distance apart, which carry a molding, above which is a plain iece of projecting wall forming a parapet, and covered by a coping. Sometimes mall arches are thrown across from corbel to corbel, to carry the projection.

Cornice. The projection at the top of a wall finished by a blocking-course, ommon in classic architecture. In Norman times, the wall finished with a corel table, which carried a portion of plain projecting work, which was finished y a coping, and the whole formed a parapet. In Early English times the paraet was much the same, but the work was executed in a much better way, espeially the small arches connecting the corbels. In the Decorated period the corbel able was nearly abandoned, and a large hollow, with one or two subordinate oldings, substituted; this is sometimes filled with the ball-flowers, and someimes with running foliages. In the Perpendicular style the parapet frequently lid not project beyond the wall-line below; the molding then became a string though often improperly called a cornice), and was ornamented by a quatre-foil, r small rosettes, set at equal intervals immediately under the battlements. In nany French examples the molded string is very bold, and enriched with foliage rnaments

Corona. The brow of the cornice which projects over the bed moldings to throw off the water.

Corridor. A long gallery or passage in a mansion connecting various apartments and running round a quadrangle. Any long passage-way in a building.

Countersink. To make a cavity for the reception of a plate of iron, or the head of a screw or bolt, so that it shall not project beyond the face of the work.

Coupled Columns. Columns arranged in pairs.

Course. A continued layer of bricks or stones in buildings; the term is also applicable to slates, shingles, etc.

Court. An open area behind a house, or in the center of a building and the wings. Courts admit of the most elegant ornamentations, such as arcades, etc.

Cove — Coving. The molding called the cavetto, or the scotia inverted, on a large scale, and not as a mere molding in the composition of a cornice, is called a cove or a coving.

Cove-bracketing. The wooden skeleton mold or framing of a cove, applied chiefly to the bracketing of a cove ceiling.

Cove Ceiling. A ceiling springing from the walls with a curve.

Coved and Flat Ceiling. A ceiling in which the section is the quadrant of a circle, rising from the walls and intersecting in a flat surface.

Cradling. Timber work for sustaining the lath and plaster of vaulted ceilings.

Cresting. An ornamental finish in the wall or ridge of a building, which is common on the Continent of Europe. An example occurs at Exeter Cathedral, the ridge of which is ornamented with a range of small fleurs-de-lis in lead.

Crocket. An ornament running up the sides of gablets, hood-molds, pinnacles, spires; generally, a winding stem like a creeping plant, with flowers or leaves projecting at intervals, and terminating in a finial.

CROCKET

Cross. This religious symbol is almost always placed on the ends of gables, the summit of spires, and other conspicuous places of old churches. In early times it was generally very plain, often a simple cross in a circle. Sometimes they take the form of a light cross, crosslet, or a cross in a square. In the Decorated and later styles they became richly floriated, and assumed an endless variety of forms. Of memorial crosses the finest examples are the Eleanor crosses, erected by Edward I. Of these a few yet remain, one of which has recently been reërected at Charing Cross. Preaching crosses were often set up by the wayside as stations for preaching; the most noted is that in front of St. Paul's, England. The finest remaining sepulchral crosses are the old elaborately carved examples found in Ireland.

Cross-aisle. An old name for a transept.

Cross-springer. The transverse ribs of a vault.

Cross-vaulting. A common name given to groins and cylindrical vaults.

Crown. In architecture the uppermost member of the cornice; called also Corona and Larmier.

Crypt. A vaulted apartment of greater or less size, usually under the choir.

Cupola. A small room, either circular or polygonal, standing on the top of a dome. By some it is called a Lantern.

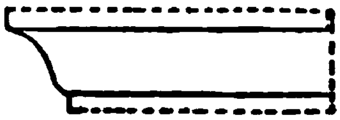
:urb **Roof,** or **Mansard Roof.** A roof formed of four contiguous planes, h two having an external inclination.

:urtail **Step.** The first step in a stair, which is generally finished in the form a scroll.

:**usp.** The point where the foliations of tracery intersect. The earliest ample in England of a plain cusp is probably that at Pythagoras School, at mbridge, of an ornamental cusp, at Ely Cathedral, where a small roll, with a ette at the end, is formed at the termination of a cusp. In the later styles the minations of the cusps were more richly decorated; they also sometimes minate not only in leaves or foliages, but in rosettes, heads, and other fanciful aments.

Cyclostyle. A structure composed of a circular range of columns without a e is cyclostylar; with a core, the range would be a peristyle. This is the spe-s of edifice called by Vitruvius monopteral.

Cyma. The name of a molding of very frequent use. It is a simple, waved e, concave at one end and convex at the other, like an lic *f*. When the concave part is uppermost it is called yma recta, but if the convexity appear above, and the acavity below, it is then a cyma reversa.



CYMA RECTA

Cymatium. When the crowning molding of an en-blature is of the cyma form, it is termed the Cyma-m.

CYMA REVERSA

Cyrtostyle. A circular projecting portico. Such are ose of the transept entrances to St. Paul's Cathedral, London.

Dado, or **Die.** The vertical face of an insulated pedestal between the base d cornice, or surbase. It is extended also to the similar part of all stereobates ich are arranged like pedestals in Roman and Italian architecture.

Dais. A part of the floor at the end of a mediæval hall, raised a step above e rest of the floor. On this the lord of the mansion dined with his friends at e great table, apart from the retainers and servants. In mediæval halls there is generally a deep recessed bay window at one or at each end of the dais, pposed to be for retirement, or greater privacy than the open hall could afford. France the word is understood as a canopy or hanging over a seat; probably e name was given from the fact that the seats of great men were then sur-ounted by such an ornament.

Darby. A flat tool used by plasterers in working, especially on ceilings. It generally about seven inches wide and forty-two inches long, with two handles the back.

Decastyle. A portico of ten columns in front.

Decorated Style. The second stage of the Pointed or Gothic style of archi-cture, considered the most complete and perfect development of Gothic archi-cture, the best examples of which are found in England.

Demi-metope. The half of a metope, which is found at the retiring or pro-cting angles of a Doric frieze.

Dentil. The cogged or toothed member, common in the bed-mold of a Corin-ian entablature, is said to be dentiled, and each cog or tooth is called a dentil.

Depressed Arches, or **Drop Arches.** Those of less pitch than the equilateral.

Design. The plans, elevations, sections, and whatever other drawings may e necessary for an edifice, exhibit the design, the term plan having a restricted oplication to a technical portion of the design.

Detail. As used by architects, detail means the smaller parts into which composition may be divided. It is applied generally to moldings and other enrichments, and again to their minutiæ.

Diameter. The line in a circle passing through its center, or thickest part, which gives the measure proportioning the intercolumniation in some of the orders.

Diameters. The diameters of the lower and upper ends of the shaft of a column are called its inferior and superior diameters, respectively; the former is the greatest, the latter the least diameter of the shaft.

Diaper. A method of decorating a wall, panel, stained glass, or any plain surface, by covering it with a continuous design of flowers, rosettes, etc., either in squares or lozenges, or some geometrical form resembling the pattern of a diapered table-cloth, from which, in fact, the name is supposed by some to have been derived.

Diastyle. A spacious intercolumniation, to which three diameters are assigned.

Dipteros. A double-winged temple. The Greeks are said to have constructed temples with two ranges of columns all around, which were called dipteroi. A portico projecting two columns and their interspaces is of dipteral or pseudo-dipteral arrangement.

Discharging Arch. An arch over the opening of a door or window, to discharge or relieve the superincumbent weight from pressing on the lintel.

Distemper. Term applied to painting with colors mixed with size or other glutinous substance. All the cartoons of the ancients, previous to the year 1410, are said to be done in distemper.

Distyle. A portico of two columns. This is not generally applied to the mere porch with two columns, but to describe a portico with two columns in antis.

Ditriglyph. An intercolumniation in the Doric order, of two triglyphs.

Dodecastyle. A portico of twelve columns in front. The lower one of the west front of St. Paul's Cathedral, London, is of twelve columns, but they are coupled, making the arrangement pseudo-dodecastyle. The Chamber of Deputies in Paris has a true dodecastyle.

Dog-tooth. A favorite enrichment used from the latter part of the Norman period to the early part of the Decorated. It is in the form of a four-leaved flower, the center of which projects, and probably was named from its resemblance to the dog-toothed violet.

Dome. A cupola or inverted cup on a building. The application of this term to its generally received purpose is from the Italian custom of calling an archepiscopal church, by way of eminence, Il Duomo, the temple; for to one of that rank, the Cathedral of Florence, the cupola was first applied in modern practice. The Italians themselves never call a cupola a dome; it is on this side of the Alps the application has arisen, from the circumstance, it would appear, that the Italians use the term with reference to those structures whose most distinguishing feature is the cupola, tholus, or (as we now call it) dome.

Domestic Architecture. That branch which relates to private buildings.

Donjon. The principal tower of a castle, generally containing the prison.

Door Frame. The surrounding case into and out of which the door shuts and opens. It consists of two upright pieces, called jambs, and a head, generally fixed together by mortices and tenons, and wrought, rebated, and beaded.

Doric Order. The oldest of the three orders of Grecian architecture.

Dormer Window. A window belonging to a room in a roof, which conse- uently projects from it with a valley gutter on each side. They are said not to e earlier than the fourteenth century. In Germany there are often several rows f dormers, one above the other. In Italian Gothic they are very rare; in fact, e former have an unusually steep roof, while in the latter country, where the :alian tile is used, the roofs are rather flat.

Dormitory. A room, suite of rooms, or building used to sleep in. The name as first applied to the place where the monks slept at night. It was sometimes ne long room like a barrack, and sometimes divided into a succession of small ambers or cells. The dormitory was generally on the first floor, and connected ith the church, so that it was not necessary to go out-of-doors to attend the octurnal services. In the large houses of the Perpendicular period, and also in me of the Elizabethan, the entire upper story in the roof formed one large partment, said to have been a place for exercise in wet weather, and also for a ormitory for the retainers of the household, or those of visitors.

Double Vault. Formed by a duplicate wall; wine cellars are sometimes so rmed.

Dovetailing. In carpentry and joinery, the method of fastening boards or ther timbers together, by letting one piece into another in the form of the xpanded tail of a dove.

Dowel. A pin let into two pieces of wood or stone, where they are joined gether. A piece of wood driven into a wall so that other pieces may be nailed it. This is also called plugging.

Draw-bridge. A bridge made to draw up or let down, much used in fortified laces. In navigable rivers, the arch over the deepest channel is made to draw r revolve, in order to let the masts of ships pass through.

Drawing-room. A room appropriated for the reception of company; a room which company withdraws from the dining-room.

Dresser. A cupboard or set of shelves to receive dishes and cooking utensils.

Dressing. Is the operation of squaring and smoothing stones for building; lso applied to smoothing lumber.

Dressing-room. An apartment appropriated for dressing the person.

Drip. A name given to the member of a cornice which has a projection eyond the other parts for throwing off water by small portions, drop by drop. t is also called Larmier.

Drip-stone. The label molding which serves on a canopy for an opening, nd to throw off the rain. It is also called Weather Molding.

Drop-scene. A curtain suspended by pulleys, which descends or drops in ont of the stage in a theater.

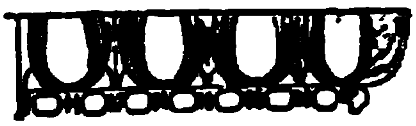
Drum. The upright part of a cupola over a dome; also, the solid part or vase f the Corinthian and Composite capitals.

Dry-rot. A rapid decay of timber, by which its substance is converted into dry powder, which issues from minute cavities resembling the borings of orms.

Dungeon. The prison in a castle keep, so called because the Norman name r the latter is donjon, and the dungeons, or prisons, are generally in its lowest ory.

Dwarf Wall. The walls enclosing courts above which are railings of iron; w walls, in general, receive this name.

Eaves. In slating and shingling, the margin or lower part of the slating hanging over the wall, to throw the water off from the masonry or brickwork.



ECHINUS

Echinus. A molding of eccentric curve, generally cut (when it is carved) into the forms of eggs and anchors alternating, whence the molding is called by the name of the more conspicuous. It is the same as Ovolo.

Edifice. Is synonymous with the terms building, fabric, erection, but is more strictly applicable to architecture distinguished for size, dignity, and grandeur.

Efflorescence. In architecture, the formation of a whitish loose powder, or crust, on the surface of stone or brick walls.

Egyptian Architecture. The earliest civilization and cultivation of the arts was in Upper Egypt. The most remarkable and most ancient monuments of the Egyptians, with the exception of the pyramids, are nearly all included in Upper Egypt. The buildings of Egypt are characterized by solidity and massiveness of construction, originality of conception, and boldness of form. The walls, the pillars, and the most sacred places of their religious buildings were ornamented with hieroglyphics and symbolical figures, while the ceilings of the porticos exhibited zodiacs and celestial planispheres. The temples of Egypt were generally without roofs, and, consequently, the interior colonnades had no pediments, supporting merely an entablature, composed of only architrave, frieze, and cornice, formed of immense blocks united without cement and ornamented with hieroglyphics.

Element. The outline of the design of a Decorated window, on which the centers for the tracery are formed. These centers will all be found to fall on points which, in some way or other, will be equimultiples of parts of the openings. To draw tracery well, or understand even the principles of its composition, much attention should be given to the study of the element.

Elevation. The front façade, as the French term it, of a structure; a geometrical drawing of the external upright parts of a building.

Embattlement. An indented parapet; battlement.

Emblazon. To adorn with figures of heraldry, or ensigns armorial.

Embossing. Sculpture in rilievo, the figures standing partly out from the plane.

Embrasure. The opening in a battlement between the two raised solid portions or merlons, sometimes called a crenelle.

Encaustic. Pertaining to the art of burning in colors, applied to painting on glass, porcelain, or tiles, where colors are fixed by heat; hence, encaustic tiles, bricks, etc.

Engaged Columns. Are those attached to, or built into walls or piers, a portion being concealed.

Enrichment. The addition of ornament, carving, etc., to plain work; decoration; embellishment.

Ensemble. Means the whole work or composition considered together, and not in parts.

Entablature. The assemblage of parts supported by the column. It consists of three parts: the architrave, frieze, and cornice.

Entail. In Gothic architecture, delicate carving.

Entasis. The swelling of a column, etc. In mediæval architecture, some pires, particularly those called "broach spires," have a slight swelling in the ides, but no more than to make them look straight; for, from a particular deceptio visus," that which is quite straight, when viewed at a height, looks ollow.

Entry. A hall without stairs or vestibule.

Epistyle. This term may with propriety be applied to the whole entablature, ith which it is synonymous; but it is restricted in use to the architrave, or west member of the entablature.

Escutcheon. (Her.) The field or ground on which a coat-of-arms is repre- nted. (Arch.) The shields used on tombs, in the spandrels of doors, or in ring-courses; also, the ornamented plates from the centre of which door rings, nockers, etc., are suspended, or which protect the wood of the key-hole from e wear of the key. In mediæval times these were often worked in a very autiful manner.

Etching. A mode of engraving on glass or metal (generally copper) by means lines, eaten in or corroded by means of some strong acid.

Eustyle. A species of intercolumniation to which a proportion of two diam- ers and a quarter is assigned. This term, together with the others of similar port — pycnostyle, systyle, diastyle, and aræostyle — referring to the distance columns from one another in composition, is from Vitruvius, who assigns to ch the space it is to express. It will be seen, however, by reference to them dividually, that the words themselves, though perhaps sufficiently applic- le convey no idea of an exactly defined space, and, by reference to the lumnar structures of the ancients, that no attention was paid by them to ch limitations. It follows, then, that the proportions assigned to each are rely conventional, and may or may not be attended to without vitiating the wer of applying the terms. Eustyle means the best or most beautiful ar- ngement; but, as the effect of a columnar composition depends on many ings besides the diameter of the columns, the same proportioned inter- lumniation would look well or ill according to those other circumstances, that the limitation of Eustyle to two diameters and a quarter is absurd.

Extrados. The exterior or convex curve forming the upper line of the arch nes; the term is opposed to the intrados, or concave side.

Eye of a Dome. The aperture at its summit.

Eye of a Volute. The circle in its center.

Façade, or **Face.** The whole exterior side of a building that can be seen at e view; strictly speaking, the principal front.

Face Mold. The pattern for marking the plank or board out of which namental hand-railings for stairs and other works are cut.

Fan Tracery. The very complicated mode of roofing used in the Perpen- cular style, in which the vault is covered by ribs and veins of tracery.

Fascia. A flat, broad member in the entablature of columns or other parts of ildings, but of small projection. The architraves in some of the orders are mposed of three bands, or fasciæ; the Tuscan and the Doric ought to have only e. Ornamental projections from the walls of brick buildings over any of the ndows, except the uppermost, are called Fasciæ.

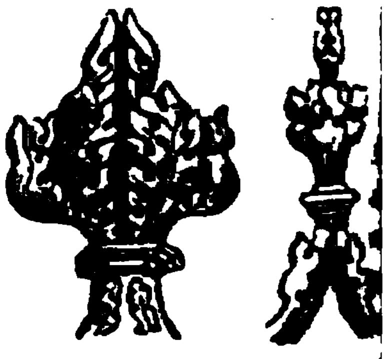
Fenestral. A frame, or "chassis," on which oiled paper or thin cloth was ained to keep out wind and rain when the windows were not glazed.

Festoon. An ornament of carved work, representing a wreath or garland of flowers or leaves, or both, interwoven with each other. It is thickest in the middle, and small at each extremity, where it is tied, a part often hanging down below the knot.

FESTOON

Fillet. A narrow vertical band or listel of frequent use in congeries of moldings, to separate and combine them, and also to give breadth and firmness to the upper edge of a crowning cyma or cavetto, as in an external cornice. The narrow slips or breadth between the flutes of Corinthian and Ionic columns are also called fillets. In mediæval work the fillet is a small, flat, projecting square, chiefly used to separate hollows and rounds, and often found in the outer parts of shafts and boutels. In this situation the center fillet has been termed a keel, and the two side ones, wings, but, apparently, this is not an ancient usage.

Finial. The flower, or bunch of flowers, with which a spire, pinnacle, gable, canopy, etc., generally terminates. Where there are crockets, the finial generally bears as close a resemblance as possible to them in point of design. They are found in early work where there are no crockets. The simplest form more resembles a bud about to burst than an open flower. They soon became more elaborate, as at Lincoln, and still more, as at Westminster and the Hôtel Cluny at Paris. Many perpendicular finials are like four crockets bound together. Almost every known example of a finial has a sort of necking separating it from the parts below



FINIALS

Fish-joint. A splice where the pieces are joined butt end to end, and are connected by pieces of wood or iron placed on each side and firmly bolted to the timbers, or pieces joined.

Flags. Flat stones, from 1 to 3 inches thick, for floors.

Flamboyant. A name applied to the Third Pointed style in France, which seems to have been developed from the Second, as the English Perpendicular was from the Decorated. The great characteristic is, that the element of the tracery flows upward in long wavy divisions like flames of fire. In most cases, also, every division has only one cusp on each side, however long the division may be. The moldings seem to be as much inferior to those of the preceding period as the Perpendicular moldings were to the Early English, a fact which seems to show that the decadence of Gothic architecture was not confined to one country.

Flange. A projecting edge, rib, or rim. Flanges are often cast on the top or bottom of iron columns, to fasten them to those above or below; the top and bottom of I-beams and channels are called the flange.

Flashings. Pieces of lead, tin, or copper, let into the joints of a wall so as to lap over gutters or other pieces; also, pieces worked in the slates or shingles around dormers, chimneys, and any rising part, to prevent leaking.

Flatting. Painting finished without leaving a gloss on the surface.

Flèche. A general term in French architecture for a spire, but more particularly used for the small, slender erection rising from the intersection of the nave and transepts in cathedrals and large churches, and carrying the sanctus bell.

Fleur-de-lis. The royal insignia of France, much used in decoration.

light. A run of steps or stairs from one landing to anotner.

loating. The equal spreading of plaster or stucco on the surface of walls, means of a board called a float; as a rule, only rough plastering is floated.

loriated. Having florid ornaments, as in Gothic pillars.

lue. The space or passage in a chimney through which the smoke ascends. ch passage is called a flue, while all together make the chimney.

lush. The continued surface, in the same plane, of two contiguous masses.

lute. A concave channel. Columns whose shafts are channeled are said be fluted, and the flutes are collectively called Flutings.

lying Buttress. An arched buttress used when extra strength was required the upper part of the wall of the nave, etc., to resist the outward thrust of a ulted ceiling. The flying buttress generally rests on the wall and buttress of aisle.

oils. The small arcs in the tracery of Gothic windows, panels, etc.

oliage. An ornamental distribution of leaves on various parts of buildings.

oliation. The use of small arcs or foils in forming tracery.

ont. The vessel used in the rite of baptism. The earliest extant is supposed be that in which Constantine is said to have been baptized; this is a porphyry rum from a Roman bath. Those in the baptisteries in Italy are all large, and re intended for immersion; as time went on, they seem to have become aller. Fonts are sometimes mere plain hollow cylinders, generally a little aller below than above; others are massive squares, supported on a thick stem, nd which sometimes there are smaller shafts. In the Early English this form still pursued, and the shafts are detached; sometimes, however, they are hex- onal and octagonal, and in this and the later styles assume the form of a vessel a stem. Norman fonts frequently have curious carvings on them, approach- the grotesque; in later times the foliages, etc., partook absolutely of the aracter of those used in other architectural details of their respective periods. e font in European churches is usually placed close to a pillar near the en- ance, generally that nearest but one to the tower in the south arcade; or, in ge buildings, in the middle of the nave, opposite the entrance porch, and netimes in a separate building. In Protestant churches in this country, the t is generally placed inside the communion rail, or on the steps of the chancel.

Footings. The spreading courses at the base or foundation of a wall. When ayer of different material from that of the wall (as a bed of concrete) is used, is called the Footing.

Foundation. That part of a building or wall which is below the surface of e ground.

Foxtail Wedging. Is a peculiar mode of mortising, in which the end of the on is notched beyond the mortise, and is split and a wedge inserted, which, ing forcibly driven in, enlarges the tenon and renders the joint firm and im- ovable.

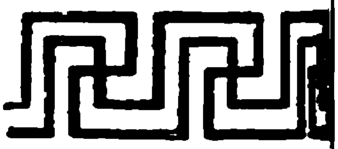
Frame. The name given to the wood-work of windows, doors, etc.; and in rpentry, to the timber works supporting floors, roofs, etc.

Framing. The rough timber work of a house, including the flooring, roofing, rtitioning, ceiling, and beams thereof.

Freestone. Stone which can be used for moldings, tracery, and other work quired to be executed with the chisel. The oölitic and sandstones are those nerally included by this term.

Fresco. The method of painting on a wall while the plastering is wet. color penetrates through the material, which, therefore, will bear rubbing or ing to almost any extent. The transparency, the chiaro-oscuro, and lucidity well as force, which can be obtained by this method, cannot be conceived the frescos of Fra Angelico or Raphael are studied. The word, however, is applied improperly to painting on the surface in distemper or body color, with size or white of egg, which gives an opaque effect.

Fret. An ornament consisting of small fillets intersecting each other at right angles.



FRET

Frieze. That portion of an entablature between the cornice above and architrave below. It derives its name from being the recipient of the sculptured enrichments either of foliage or figures which may be relevant to the object of the sculpture. The frieze is also called the Zoopho

Frigidarium. An apartment in the Roman bath, supplied with cold water

Furniture. A name given to the metal trimmings of doors, windows, other similar parts of a house. In this country the word "hardware" is generally used to denote the same thing.

Furrings. Flat pieces of timber used to bring an irregular framing to an surface.

Gable. When a roof is not hipped or returned on itself at the ends, its are stopped by carrying up the walls under them in the triangular form of roof itself. This is called the gable, or, in the case of the ornamental and mented gable, the pediment. Of necessity, gables follow the angles of the of the roof, and differ in the various styles. In Norman work they are gener about half-pitch; in Early English, seldom less than equilateral, and often In Decorated work they become lower, and still more so in the Perpendicul style. In all important buildings they are finished with copings or parapets. the Later Gothic styles gables are often surmounted with battlements, or enrich with crockets; they are also often paneled or perforated, sometimes very rich The gables in ecclesiastical buildings are mostly terminated with a cross; others, by a finial or pinnacle. In later times the parapets or copings were brok into a sort of steps, called corbic steps. In buildings of less pretension the or other roof covering passed over the front of the wall, which then, of cour had no coping. In this case, the outer pair of rafters were concealed by mold or carved verge boards.

Gable Window. A term sometimes applied to the large window under a gab but more properly to the windows in the gable itself.

Gabled Towers. Those which are finished with gables instead of parapet Many of the German Romanesque towers are gabled.

Gablets. Triangular terminations to buttresses, much in use in the Ea English and Decorated periods, after which the buttresses generally terminate pinnacles. The Early English gablets are generally plain, and very sharp pitch. In the Decorated period they are often enriched with paneling and crockets. They are sometimes finished with small crosses, but oftener with finials.

Gain. A beveled shoulder on the end of a mortised brace, for the purpose giving additional resistance to the shoulder.

Gallery. Any long passage looking down into another part of a building into the court outside. In like manner, any stage erected to carry a rood or organ, or to receive spectators, was latterly called a gallery, though originally

. In later times the name was given to any very long rooms, particularly se intended for purposes of state, or for the exhibition of pictures.

ambrel Roof. A roof with two pitches, similar to a mansard or curb roof.

argoyle, or Gurgoyle. The carved termination spout which conveyed away the water from the ters, supposed to be called so from the gurgling e made by the water passing through it. Gar- les are mostly grotesque figures.

ate-house. A building forming the entrance to own, the door of an abbey, or the enceinte of a le or other important edifice. They generally had rge gateway protected by a gate, and also a port- is, over which were battlemented parapets with s (machicolations) for throwing down darts, melted l, or hot sand on the besiegers. Gate-houses ays had a lodge, with apartments for the porter, . guard-rooms for the soldiers; and, generally, ms over for the officers, and often places for prisoners beneath. The name is r commonly applied to the gate-keeper's lodge on large estates.

GARGOYLE

auge. To mix plaster of Paris with common plaster to make it set quick, ed gauged mortar. A tool used by carpenters, to strike a line parallel to the e of a board.

irder. A large timber or iron beam, either single or built up, used to sup- t joists or walls over an opening.

lyph. A vertical channel in a frieze.

othic Style. The name of Gothic was given to the various Mediæval styles a period in the sixteenth century when a great classic revival was going on, l everything not classic was considered barbarian, or Gothic. The term was s originally intended as one of stigma, and, although it conveys a false idea of character of the Mediæval styles, it has long been used to distinguish them m the Grecian and Roman. The true principle of Gothic architecture is the tical division, relation and subordination of the different parts, distinct and at unity with each other, and while this principle was adhered to, Gothic hitecture may be said to have retained its vitality.

range. A word derived from the French, signifying a large barn or granary. anges were usually long buildings with high wooden roofs, sometimes divided posts or columns into a sort of nave and aisles, with walls strongly buttressed. England the term was applied not only to the barns, but to the whole of the ldings which formed the detached farms belonging to the monasteries; in st cases there was a chapel either included among these or standing apart as a arate edifice.

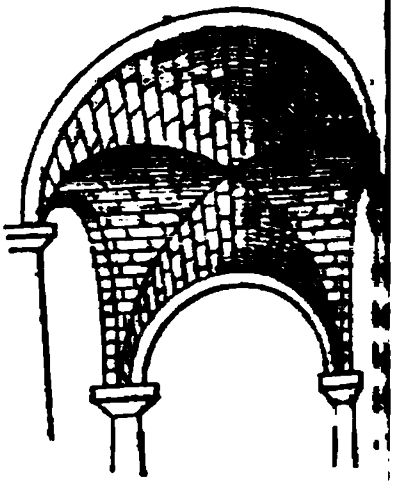
rillage. A framework of beams laid longitudinally and crossed by similar ms notched upon them, used to sustain walls to prevent irregular settling.

rille. The iron-work forming the enclosure screen to a chapel, or the pro- ting railing to a tomb or shrine; more commonly found in France than in gland. They are of wrought iron, ornamented by the swage and punch, and t together either by rivets or clips. In modern times grilles are used exten- ely for protecting the lower windows in city houses, also the glass opening in tside doors.

roin. By some described as the line of intersection of two vaults where they et each other, which others call the groin point; by others the curved section

or spandrel of such vaulting is called a groin, and by others the whole syst vaulting is so named.

Groin Arch. The cross-rib in the later styles of groining, passing at right angles from wall to wall, and dividing the vault into bays or travees.



GROINED VAULTING

Groin Ceiling. A ceiling to a building composed of oak ribs, the spandrels of which are filled in with narrow, thin slips of wood. There are several in England; one at the Early English church at Warmington, and one at Winchester Cathedral, exactly resembling those of stone.

Groin Centring. In groining without ribs, the whole surface is supported by centring during the erection of the vaulting. In ribbed work the stone ribs only are supported by timber ribs during the progress of the work, any light stuff being used while filling in the spand

Groin Point. The name given by workmen to the arris or line of intersecti of one vault with another where there are no ribs.

Groin Rib. The rib which conceals the groin point or joints, where the sp drels intersect.

Groined Vaulting. The system of covering a building with stone va which cross and intersect each other, as opposed to the barrel vaulting, or ser of arches placed side by side. The earliest groins are plain, without any ri except occasionally a sort of wide band from wall to wall, to strengthen the con struction. In later Norman times ribs were added on the line of intersection the spandrels, crossing each other, and having a boss as a key common to bo these ribs the French authors call *nerfs en ogive*. Their introduction, howev caused an entire change in the system of vaulting; instead of arches of unif thickness and great weight, these ribs were first put up as the main constructi and spandrels of the lightest and thinnest possible material placed upon them, haunches only being loaded sufficiently to counterbalance the pressure from crown. Shortly after, half-ribs against the walls (formerets) were introduced carry the spandrels without cutting into the walling, and to add to the appeara The work was now not treated as continued vaulting, but as divided into ba and it was formed by keeping up the ogive, or intersecting ribs and their bos a sort of construction having some affinity to the dome was formed, which ad much to the strength of the groining. Of course, the top of the soffit or ridge the vault was not horizontal, but rose from the level of the top of the formerets to the boss and fell again; but this could not be perceived from below. As t system of construction got more into use, and as the vaults were required to be greater span and of higher pitch, the spandrels became larger, and required m support. To give this, another set of ribs was introduced, passing from springers of the ogive ribs, and going to about half-way between these and ogive, and meeting on the ridge of the vault; these intermediate ribs are ca by the French *tiercerons*, and began to come into use in the transition from E English to Decorated. About the same period a system of vaulting came i use called *hexpartite*, from the fact that every bay is divided into six comp ments instead of four. It was invented to cover the naves of churches of unu width. The filling of the spandrels in this style is very peculiar, and, wher different compartments meet at the ridge, some pieces of harder stone ha been used, which give rather a pleasing effect. The arches against the wa being of smaller span than the main arches, cause the centre springers to be

dicular and parallel for some height, and the spandrels themselves are very ow. As styles progressed, and the desire for greater richness increased, ther series of ribs, called *liernes*, was introduced; these passed crossways from *ogives* to the *tiercerons*, and thence to the *doubleaux*, dividing the spandrels rly horizontally. These various systems increased in the Perpendicular od, so that the walls were quite a net-work of ribs, and led at last to the lor, or, as it is called by many, fan-tracery vaulting. In this system the ribs no part of the real construction, but are merely carved upon the voussoirs, ch form the actual vaulting. Fan Tracery is so called because the ribs iate from the springers, and spread out like the sticks of a fan. These later hods are not strictly groins, for the pendentives are not square on plan, but ular, and there is, therefore, no arris intersection or groin point.

roins, Welsh, or **Underpitch.** When the main longitudinal vault of any ning is higher than the cross or transverse vaults which run from the windows, system of vaulting is called underpitch groining, or, as termed by the workn, Welsh groining. A very fine example is at St. George's Chapel, Windsor, gland.

roove. In joinery, a term used to signify a sunk channel whose section is angular. It is usually employed on the edge of a molding, stile, or rail, , into which a tongue corresponding to its section, and in the substance of wood to which it is joined, is inserted.

rotesque. A singular and fantastic style of ornament found in ancient ldings.

rotto. An artificial cavern.

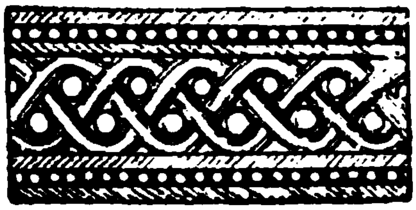
round Floor. The floor of a building on a level, or nearly so, with the und.

round Joist. Joist that is blocked up from the ground.

rounds. Pieces of wood embedded in the plastering of walls to which ting and other joiner's work is attached. They are also used to stop the tering around door and window openings.

rouped Columns. Three, four, or more columns put together on the same estal. When two are placed together, they are said to be coupled.

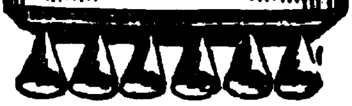
rout. Mortar made so thin by the addition of er that it will run into all the joints and cavities he mason-work, and fill it up solid.



GUILLOCHE

uilloche, or Guillochos. An interlaced ornat like net-work, used most frequently to enrich torus.

uttæ. The small cylindrical drops used to en- the mutules and regulæ of the Doric entabla- are so called.



GUTTÆ

utter. The channel for carrying off rain-water. mediæval gutters differed little from others, except t they are often hollows sunk in the top of stone nices, in which case they are generally called chan- in English, and *cheneaux* in French.

ymnasium. A building classed in the first rank by the Greeks; it was in m they instructed the youth in all the arts of peace and war; a building for letic exercises.

Hall. The principal apartment in the large dwellings of the Mi used for the purposes of receptions, feasts, etc. In the Norman cas was generally in the keep above the ground floor, where the retainer basement being devoted to stores and dungeons for confining prison halls — indeed, some Norman halls (not in castles) — are generally on floor, as at Westminster, approached by a porch either at the end, as example, or at the side, as at Guildhall, London, having at one end a or estrade. The roofs are generally open and more or less ornament middle of these was an opening to let out the smoke, though in later halls have large chimney-places with funnels or chimney-shafts for th At this period there were usually two deeply recessed bay windows of the dais, and doors leading into the withdrawing-rooms, or the b ments; they are also generally wainscoted with oak, in small panels, to of five or six feet, the panels often being enriched. Westminster originally divided into three parts, like a nave and side aisles, as are Continent of Europe. A room or passage-way at the entrance of a suite of chambers. A place of public assembly, as a town-hall, a m

Halving. The junction of two pieces of timber, by letting one into

Hammer Beam. A beam in a Gothic roof, not extending to th side; a beam at the foot of a rafter.

Hanging Buttress. A buttress not rising from the ground, but sup a corbel, applied chiefly as a decoration and used only in the Deco Perpendicular style.

Hanging Stile. Of a door, is that to which the hinges are fixed.

Hangings. Tapestry; originally invented to hide the coarseness of of a chamber. Different materials were employed for this purpose, them exceedingly costly and beautifully worked in figures, gold and s

Hatching. Drawing parallel lines close together for the purpose o ing a section of anything. The lines are generally drawn at an angle of a horizontal.

Haunches. The sides of an arch, about half-way from the springing crown.

Headers. In masonry, are stones or bricks extending over the thickness of a wall. In carpentry, the large beam into which the common joists are framed in framing openings for stairs, chimneys, etc.

Heading Courses. Courses of a wall in which the stone or brick are all headers.

Head-way. Clear space or height under an arch, or over a stairway, and the like.

Heel. Of a rafter, the end or foot that rests upon the wall plate.

Height. Of an arch, a line drawn from the middle of the chord to the intrados.

Helix. A small volute or twist like a stalk, representing the twisted tops of the acanthus, placed under the abacus of the Corinthian capital.

Hermes. A rough quadrangular stone or pillar, having a head, usually of Hermes or Mercury, sculptured on the top, without arms or body, placed by the Greeks in front of buildings.

HERMES

ing-bone Work. Bricks, tile, or other materials arranged diagonally ling.

astyle. A portico of six columns in front is of this description.

Altar. The principal altar in a cathedral or church. Where there is a it is generally at the end of the choir or chancel, not in the lady chapel.

knob. The finial on the hip of a roof, or between the barge boards of a

roof. A roof which rises by equally inclined planes from all four sides building.

podrome. A place appropriated by the ancients for equestrian exercises.

s. Those pieces of timber placed in an inclined position at the corners or of a hip-roof.

d-mold. A word used to signify the drip-stone for label over a window r opening, whether inside or out.

el de Ville. The town-hall, or guild-hall, in France, Germany, and ern Italy. The building, in general, serves for the administration of justice, ceipt of town dues, the regulation of markets, the residence of magistrates, ks for police, prisons, and all other fiscal purposes. As may be imagined, liffer very much in different towns, but they have almost invariably ed to them, or closely adjacent, a large clock-tower containing one or bells, for calling the people together on special occasions.

tel Dieu. The name for a hospital in mediæval times. In England there it few remains of these buildings, one of which is at Dover; in France there any. The most celebrated is the one at Angers, described by Parker. do not seem to differ much in arrangement of plan from those in modern the accommodation for the chaplain, medicine, nurses, stores, etc., being the same in all ages, except that in some of the earlier, instead of the sick placed in long wards like galleries, as is now done, they occupied large ings, with naves and side aisles, like churches.

using. The space taken out of one solid to admit the insertion of another. ase on a stair is generally housed into the treads and risers; a niche for a e.

pæthros. A temple open to the air, or uncovered. The term may be the easily understood by supposing the roof removed from over the nave of a h in which columns or piers go up from the floor to the ceiling, leaving the still covered.

pogea. Constructions under the surface of the earth, or in the sides of a r mountain.

nography. A horizontal section of a building or other object, showing its dimensions according to a geometric scale, a ground plan.

pluvium. The central part of an ancient Roman court, which was unred.

post. A term in classic architecture for the horizontal moldings of piers ilasters, from the top of which spring the archivolts or moldings which go d the arch.

Antis. When there are two columns between the antæ of the lateral walls the cella.

cise. To cut in; to carve; to engrave.

dented. Toothed together.

Inlaying. Inserting pieces of ivory, metal, or choice woods, or the like, in a groundwork of some other material, for ornamentation.

Insulated. Detached from another building. A church is insulated, when not contiguous to any other edifice. A column is said to be insulated, when standing free from the wall; thus, the columns of peripteral temples were insulated.

Intaglio. A sculpture or carving in which the figures are sunk below the general surface, such as a seal the impression of which in wax is in bas-relief; opposed to Cameo.

Intercolumniation. The distance from column to column, the clear space between columns.

Interlaced Arches. Arches where one passes over two openings, and they consequently cut or intersect each other.

Intrados. Of an arch, the inner or concave curve of the arch stones.

Inverted Arches. Those whose key-stone or brick is the lowest in the arch.

Ionic Order. One of the orders of Classical architecture.

Iron Work. In mediæval architecture, as an ornament, is chiefly confined to the hinges, etc., of doors and of church chests, etc. In some instances not only do the hinges become a mass of scroll work, but the surface of the doors is covered by similar ornaments. In almost all styles the smaller and less important doors had merely plain strap hinges, terminating in a few bent scrolls, and latterly in fleurs-de-lis. Escutcheon and ring handles, and the other furniture, partake more or less of the character of the time. On the Continent of Europe the knockers are very elaborate. At all periods doors have been ornamented with nails having projecting heads, sometimes square, sometimes polygonal, and sometimes ornamented with roses, etc. The iron work of windows is generally plain and the ornament confined to simple fleur-de-lis heads to the stanchions. The iron work of screens enclosing tombs and chapels is noticed under *Grille*, q.v.

Jack. An instrument for raising heavy loads, either by a crank, screw and pinion, or by hydraulic power, and in all cases worked by hand.

Jack Rafter. A short rafter, used especially in hip-roofs.

Jamb. The side-post or lining of a doorway or other aperture. The jambs of a window outside the frame are called Reveals.

Jamb-shafts. Small shafts to doors and windows with caps and bases; when in the inside arris of the jamb of a window they are sometimes called Escoinsons.

Joggle. A joint between two bodies so constructed by means of jogs or notches as to prevent their sliding past each other.

Joinery. That branch in building confined to the nicer and more ornamental parts of carpentry.

Joist. A small timber to which the boards of a floor or the laths of ceilings are nailed. It rests on the wall or on girders.

Keep. The inmost and strongest part of a mediæval castle, answering to the citadel of modern times. The arrangement is said to have originated with Gundulf, the celebrated Bishop of Rochester. The Norman keep is generally a very massive square tower, the basement or stories partly below ground being used for stores and prisons. The main story is generally a great deal above ground level, with a projecting entrance, approached by a flight of steps and drawbridge. This floor is generally supposed to have been the guard-room or place for the soldiery; above this was the hall, which generally extended over the whole area of the building, and is sometimes separated by columns; above this are other apartments for the residents. There are winding staircases in the angles of

uildings, and passages and small chambers in the thickness of the walls. The eep was intended for the last refuge, in case the outworks were scaled and the ther buildings stormed. There is generally a well in a mediæval keep, ingenusly concealed in the thickness of a wall, or in a pillar. The most celebrated f Norman times are the White Tower in London, the castles at Rochester, rundel, and Newcastle, Castle Hedingham, etc. The keep was often circular.

Key-stone. The stone placed in the center of the top of an arch. The charcter of the key-stone varies in different orders. In the Tuscan and Doric it is nly a simple stone projecting beyond the rest; in the Ionic it is adorned with oldings in the manner of a console; in the Corinthian and Composite it is a ch-sculptured console.

King-post. The middle post of a trussed piece of framing for supporting the e-beam at the middle and the lower ends of the struts.

Knee. A piece of timber naturally or artificially bent to receive another to lieve a weight or strain.

Knob, Knot. The bunch of flowers carved on a corbel, or on a Boss.

Kremlin. The Russian name for the citadel of a town or city.

Label. Gothic: the drip or hood-molding of an arch, when it is returned to e square.

Label Terminations. Carvings on which the labels terminate near the pringing of the windows. In Norman times those were frequently grotesque eads of fish, birds, etc., and sometimes stiff foliage. In the Early English and ecorated periods they are often elegant knots of flowers, or heads of kings, ueens, bishops, and other persons supposed to be the founders of churches. In e Perpendicular period they are often finished with a short square, mitred return knee, and the foliages are generally leaves of square or octagonal form.

Lacunar. A paneled or coffered ceiling or soffit. The panels or cassoons of ceiling are by Vitruvius called lacunaria.

Lady-chapel. A small chapel dedicated to the irgin Mary, generally found in ancient cathedrals.

Lancet. A high and narrow window pointed like lancet, often called a lancet window.

Landing. A platform in a flight of stairs between o stories; the terminating of a stair.

LACUNARS IN CEILING

Lantern. A turret raised above a roof or tower d very much pierced, the better to transmit light. modern practice this term is generally applied to y raised part in a roof or ceiling containing vertical ndows, but covered in horizontally. The name was also often applied to the uver or femerell on a roof to carry off the smoke; sometimes, too, to the open nstructions at the top of towers, as at Ely Cathedral, probably because lights re placed in them at night to serve as beacons.

Lanterns of the Dead. Curious small slender towers, found chiefly in the ntre and west of France, having apertures at the top, where a light was expited at night to mark the place of a cemetery. Some have supposed that the and towers in Ireland may have served for this purpose.

Lath. A slip of wood used in slating, tiling, and plastering.

Lattice. Any work of wood or metal made by crossing laths, rods, or bars, d forming a net-work. A reticulated window, made of laths or slips of iron,

separated by glass windows, and only used where air rather than light is to be admitted, as in cellars and dairies.

Lavabo. The lavatory for washing hands, generally erected in cloisters of monasteries. A very curious one at Fontenay, surrounding a pillar, is given by Viollet-le-Duc. In general, it is a sort of trough, and in some places has an almry for towels, etc.

Lavatory. A place for washing the person.

Lean-to. A small building whose rafters pitch or lean against another building, or against a wall.

Lectern. The reading-desk in the choir of churches.

Ledge, or Ledgement. A projection from a plane, as slips on the side of window and door frames to keep them steady in their places.

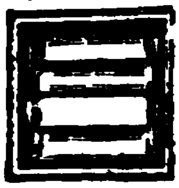
Ledgers. The horizontal pieces fastened to the standard poles or timbers of scaffolding raised around buildings during their erection. Those which rest on the ledgers are called putlogs, and on these the boards are laid.

Lewis. An iron clamp dovetailed into a large stone to lift it by.

Lich-gate. A covered gate at the entrance of a cemetery, under the shelter of which the mourners rested with the corpse, while the procession of the clergy came to meet them. There are several examples in England.

Light. A division or space in a sash for a single pane of glass; also a pane of glass.

Linen Scroll. An ornament formerly used for filling panels, and so called from its resemblance to the convolutions of a folded napkin.



LINEN SCROLL

Lining. Covering for the interior, as casing is covering for the exterior surface of a building; also, such as linings of a door for windows, shutters, and similar work.

Lintel. The horizontal piece which covers the opening of a door or window.

Lip Mold. A molding of the Perpendicular period like a hanging lip.

List, or Listel. A little square molding, to crown a larger; also termed a fillet.

Lithograph. A print from a drawing on stone.

Lobby. An open space surrounding a range of chambers, or seats in a theater; a small hall or waiting room.

Lodge. A small house in a park.

Loft. The highest room in a house, particularly if in the roof; also, a gallery raised up in a church to contain the rood, the organ, or singers.

Loggia. An outside gallery or portico above the ground, and contained within the building.

Loop-hole. An opening in the wall of a building, very narrow on the outside and splayed within, from which arrows or darts might be discharged on an enemy. They are often in the form of a cross, and generally have round holes at the ends.

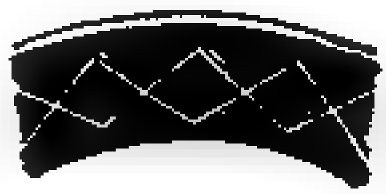
Lombard Architecture. A name given to the round-arched architecture of Italy, introduced by the conquering Goths and Ostrogoths, and which superseded the Romanesque. It reigned between the eighth and twelfth centuries during the time that the Saxon and Norman styles were in vogue in England, and corresponded with them in its development into the Continental Gothic.

Lotus. A plant of great celebrity amongst the ancients, the leaves and lossoms of which generally form the capitals of Egyptian columns.

Louver. A kind of vertical window, frequently in the peaks of gables, and in he top of towers, and provided with horizontal slats which ermit ventilation and exclude rain.

Lozenge Molding. A kind of molding used in Norman rchitecture, of many different forms, all of which are charcterized by lozenge-shaped ornaments.

Lunette. The French term or the circular opening in the roining of the lower stories of owers, through which the bells re drawn up.



LOZENGE MOLDING

LOUVER WINDOW

Machicolation. A parapet r gallery projecting from the upper part of the wall of a house or fortificaion, supported by brackets or corbels, and perforated in the lower part so that he defenders of the building might throw down darts, stones, and sometimes ot sand, molten lead, etc., upon their assailants below.

Man-hole. A hole through which a man may creep into a drain, cesspool, team-boiler, etc.

Manor-house. The residence of the suzerain or lord of the manor; in France e central tower or keep of a castle is often called the *manoir.*

Mansard Roof. Curb roof, invented by François Mansard, a distinguished rench architect, who died in 1666.

Mansion. A residence of considerable size and pretension.

Mantel. The work over a fireplace in front of a chimney; especially, a shelf, sually ornamented, above the fireplace.

Marquetry. Inlaid work of fine hard pieces of wood f different colors, also of shells, ivory, and the like.

Mausoleum. A magnificent tomb or sumptuous sepulhral monument.

Medallion. Any circular tablet on which are embossed gures or busts.

MACHICOLATION

Mediæval Architecture. The architecture of England, France, Germany, etc., during the Middle Ages, including the Norman and arly Gothic styles. It comprises also the Romanesque, Byzantine and Saraenic, Lombard, and other styles.

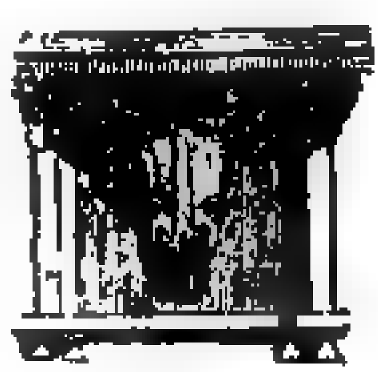
Members. The different parts of a building, the different parts of an entabtture, the different moldings of a cornice, etc.

Merlon. That part of a parapet which lies between wo embrasures.

Metope. The square recess between the triglypus in Doric frieze. It is sometimes occupied by sculptures.

Mezzanine. A low *story* between two lofty ones. t is called by the French *entresol*, or inter-story.

Mezzo-rilievo. Or mean relief, in comparison with lto-rilievo, or high relief.

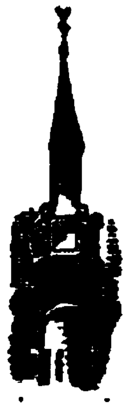


METOPE

Minaret. Turkish: a circular turret rising by different stages or divisions, each of which has a balcony.

Minster. Probably a corruption of monasterium — the large church attached to any ecclesiastical fraternity. If the latter be presided over by a bishop, it is generally called a Cathedral; if by an abbot, an Abbey; if by a prior, a Priory.

Minute. The sixtieth part of the lower diameter of a column; it is the measure used by architects to determine the proportions of an order.



MINARET

Miserere. A seat in a stall of a large church made to turn up and afford support to a person in a position between sitting and standing. The under side is generally carved with some ornament, and very often with grotesque figures and caricatures of different persons.

Miter. A molding returned upon itself at right angles is said to miter. In joinery, the ends of any two pieces of wood of corresponding form, cut off at 45°, necessarily abut upon one another so as to form a right angle, and are said to miter.

Modillion. So called because of its arrangement in regulated distances; the enriched block or horizontal bracket generally found under the cornice of the Corinthian entablature. Less ornamented, it is sometimes used in the Ionic.

MODILLION

Module. This is a term which has been generally used by architects in determining the relative proportions of the various parts of a columnar ordinance. The semi-diameter of the column at its base is the module, which being divided into thirty parts called minutes, any part of the composition is said to be of so many modules and minutes, or minutes alone, in height, breadth, or projection. The whole diameter is now generally preferred as a module, it being a better rule of proportion than its half.

Monastery. A set of buildings adapted for the reception of any of the various orders of monks, the different parts of which are described in the separate article, *Abbey*.

Monotriglyph. The intercolumniations of the Doric order are determined by the number of triglyphs which intervene, instead of the number of diameters of the column, as in other cases; and this term designates the ordinary intercolumniation of one triglyph.

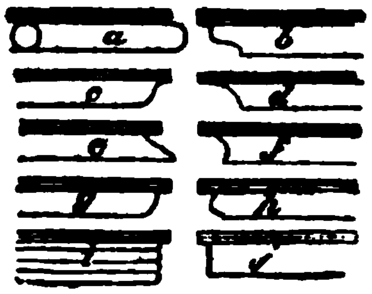
Monument. A name given to a tomb, particularly to those fine structures recessed in the walls of mediæval churches.

Mosaic. Pictorial representations, or ornaments, formed of small pieces of stone, marble, or enamel of various colors. In Roman houses the floors are often entirely of mosaic, the pieces being cubical. The best examples of mosaic work are found in St. Mark's, at Venice.

Mosque. A Mahometan temple, or place of worship.

Molding. When any work is wrought into long regular channels or projections, forming curves or rounds, hollows, etc., it is said to be molded, and each separate member is called a molding. In mediæval architecture the principal moldings are those of the arches, doors, windows, piers, etc. In the Early English style, the moldings, for some time, formed groups set back in square, and frequently very deeply undercut. The scroll molding is also common

all fillets now become very frequent in the keel molding, from its resemblance ection to the bottom of a ship; sometimes, also, it has a peculiar hollow on h side, like two wings. Later in the Decorated le the moldings are more varied in design, though lows and rounds still prevail. The undercutting is so deep, fillets abound, ogees are more frequent, the wave mold, double ogee, or double ressaunt, is n seen. In many places the strings and labels are und, the lower half of which is cut off by a plain mfer. The moldings in the later styles in some ree resemble those of the Decorated, flattened and nded; they run more into one another, having er fillets, and being, as it were, less grouped. of the principal features of the change is the stitution of one, or perhaps two (seldom more), large hollows in the set of moldings. These hol- s are neither circular nor elliptical, but obovate, an egg cut across, so that one half is larger the other. The brace mold also has a small l, where the two ogees meet. Another sort of ding, which has been called a lip mold, is common in parapets, bases, and therings.



MOLDINGS

a, astragal; *b*, ogee; *c*, cymatium; *d*, cavetto; *e*, scotia, or casement; *f*, apophyges; *g*, ovolo, or quarter round; *h*, torus; *i*, reeding; *j*, band.

Ioldings, Ornamented. The Saxon and early Norman moldings do not n to have been much enriched, but the complete and later styles of Norman remarkable for a profusion of ornamentation, the most usual of which is t is called the zigzag. This seems to be to Norman architecture what the nder or fret was to the Grecian; but it was probably derived from the ons, as it is very frequently found in their pottery. Bezants, quatrefoils, ages, crescents, billets, heads of nails, are very common ornaments. Besides e, battlements, cables; large ropes round which smaller ropes are turned, or, ur sailors say, "wormed"; scallops, pellets, chains, a sort of conical barrels, nt stiff foliages, beaks of birds, heads of fishes, ornaments of almost every con- ible kind, are sculptured in Norman moldings; and they are used in such usion as has been attempted in nc other style. The decorations on Early ish moldings are chiefly the dog-tooth, which is one of the great charac- tics of this style, though it is to be found in the Transition Norman. It is rally placed in a deep hollow between two projecting moldings, the dark ow in the hollow contrasting in a very beautiful way with the light in these ings. In this period and in the next the tympanum over doorways, par- urly if they are double doors, is highly ornamented. Those of the Decorated d resemble the former, except that the foliage is more natural and the dog- gives way to the ball-flower. Some of the hollows, also, are ornamented rosettes set at intervals, which are sometimes connected by a running tendril, e ball-flowers are frequently. Some very pleasing leaf-like ornaments in the s of windows are often found in Continental architecture. In the Perpen- ar period the moldings are ornamented very frequently by square four- d flowers set at intervals, but the two characteristic ornaments of the time unning patterns of vine leaves, tendrils, and grapes in the hollows, which d writers are called "vignettes in casements," and upright stiff leaves, ally called the Tudor leaf. On the Continent moldings partook much of ame character.

illion, Munion. The perpendicular pieces of stone, sometimes like col- , sometimes like slender piers, which divide the bays or lights of windows

o · screen-work from each other. In all styles, in less important work, the m lions are often simply plain chamfered, and more commonly have a very fla low on each side. In larger buildings there is often a bead or boutell on the and often a single very small column with a capital. As tracery grew richer, windows were divided by a larger order of mullion, between which came a l or subordinate set of mullions, which ran into each other. The term is applied to a wood or iron division between two windows.

Multifoil. A leaf ornament consisting of more than five divisions, applied foils in windows.

Mutule. The rectangular impending block under the corona of the Do cornice, from which guttæ or drops depend. Mutule is equivalent to modil but the latter term is applied more particularly to enriched blocks or bracke such as those of Ionic and Corinthian entablatures.

Narthex. The long arcaded porch forming the entrance into the Christi basilica. Sometimes there was an inner narthex, or lobby, before entering t church. When this was the case, the former was called exo-narthex, and t latter eso-narthex. In the Byzantine churches this inner narthex forms part the solid structure of the church, being marked off by a wall or row of colum whereas in the Latin churches it was usually formed only by a wooden or ot temporary screen.

Natural Beds. In stratified rocks, the surface of a stone as it lies in t quarry. If not laid in walls in their natural bed the laminæ separate.

Nave. The central part between the arches of a church, which formerly w separated from a chancel or choir by a screen. It is so called from its fanci resemblance to a ship. In the nave were generally placed the pulpit and fo In continental Europe it often also contains a high altar, but this is of ra occurrence in England.

Necking. The annulet or round, or series of horizontal moldings, whi separate the capital of a column from the plain part or shaft.

Newel. In mediæval architecture, the circular ends of a winding stairca which stand over each other, and form a sort of cylindrical column.

Newel Post. The post, plain or ornamented, placed at the first, or la step, to receive or start the hand-rail upon.

Niche. A recess sunk in a wall, generally for the reception of a sta Niches sometimes terminate by a simple label, but more commonly by a can and with a bracket or corbel for the figure, in which case they are often ca tabernacles.

Norman Style. Was that species of Romanesque which was practised by t Normans, and which was introduced and fully developed in England after th had established themselves in it. The chief features of this style are pla and massiveness. The arches, windows, and doorways were semicircular. pillars were very massive, and often built up of small stones laid like brick

Nosings. The rounded and projecting edges of the treads of a stair, or edge of a landing.

Obelisk. Lofty pillars of stone, of a rectangular form, diminishing towa top, and generally ornamented with inscriptions and hieroglyphics amo ancient Egyptians.

Observatory. A building erected on an elevated spot of ground for ma astronomical observations.

Octostyle. A portico of eight columns in front.

)ffsets. When the face of a wall is not one continued surface, but sets in by izontal jogs, as the wall grows higher and thinner, the jogs are called offsets.

)gee. The name applied to a molding, partly a hollow and partly a round, derived no doubt from its resemblance to an O placed over a G. It is rarely ıd in Norman work, and is not very common in Early English. It is of fre-nt use in Decorated work, where it becomes sometimes double, and is called a /e molding; and later still, two waves are connected with a small bead, which hen called a brace molding. In ancient MSS. it is called a Ressaunt.

)rchestra. In ancient theaters, where the chorus used to dance; in modern ıters, where the musicians sit.

)rder. A column with its entablature and stylobate is so called. The term is result of the dogmatic laws deduced from the writings of Vitruvius, and has ı exclusively applied to those arrangements which they were thought to rant.

)riel Window. Gothic: a projecting angular window, commonly of a tri-ıal or pentagonal form, and divided by mullions and transoms into different s and compartments.

)rthography. A geometrical elevation of a building or other object in which represented as it actually exists or may exist, and not perspectively, or as it ıld appear.

)rthostyle. A columnar arrangement in which the columns are placed in a ight line.

)volo. Same as *Echinus*.

'agoda. A name given to temples in India and China.

'alace. The dwelling of a king, prince, or bishop.

'ale. A fence picket, sharpened at the upper end.

'ane. Probably a diminutive of panneau, a term applied to the different es of glass in a window; same as *Light*.

'anel. Properly a piece of wood framed within four other pieces of wood, as he styles and rails of a door, filling up the aperture, but often applied both to whole square frame and the sinking itself; also to the ranges of sunken com-ments in wainscoting, cornices, corbel tables, groined vaults, ceilings, etc.

antograph, or **Pentagraph.** An instrument for copying on the same, or an rged or reduced scale.

antry. An apartment or closet in which bread and other provisions are t.

apier-maché. A hard substance made of a pulp from rags or paper mixed ı size or glue, and molded into any desired shape. Much used for architec-l ornaments.

arapet. A dwarf wall along the edge of a roof, or round a terrace walk, etc., revent persons from falling over, and as a protection to the defenders in case siege. Parapets are either plain, embattled, perforated, or paneled. The two are found in all styles except the Norman. Plain parapets are simply ions of the wall generally overhanging a little, with coping at the top and el table below. Embattled parapets are sometimes paneled, but oftener ed for the discharge of arrows, etc. Perforated parapets are pierced in ous devices — as circles, trefoils, quatrefoils, and other designs — so that the is seen through. Paneled parapets are those ornamented by a series of ls, eitner oblong or square, and more or less enriched, but are not perforated. e are common in the Decorated and Perpendicular periods.

Pargeting. A species of plastering decorated by impressing patterns when wet. These seem generally to have been made by sticking a number of pins in a board in certain lines or curves, and then pressing on the wet plaster in various directions, so as to form geometrical figures. Sometimes these devices are in relief, and in the time of Elizabeth represent figures, birds, foliage, etc. Rough plastering, commonly adopted for the interior surface of chimneys.

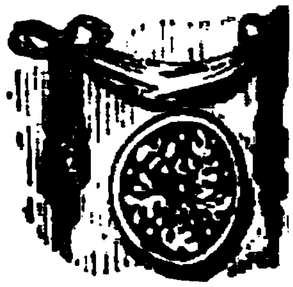
Parlor. A room in a house which the family usually occupy for society and conversation, and for receiving visitors. The apartment in a monastery or nunnery where the inmates are permitted to meet and converse with each other, or with visitors and friends from without.

Parochial. Belonging or relating to a parish.

Parquetry, or **Marquetry.** A kind of inlaid floor composed of small pieces of wood either square or triangular, which are capable of forming, by their position, various combinations of figures; this description of joinery is very suitable for the floors of libraries, halls, and public apartments.

Party Walls. Partitions of brick or stone between buildings on two adjoining properties.

Patera. A circular ornament resembling a dish, often worked in relief on friezes, etc.



PATERA

Pavement. Tessellated, a pavement of mosaic work, used by the ancients, made of square pieces of stone, etc., called Tessera.

Pavilion. A turret or small insulated building, and comprised beneath a single roof; also, the projecting part in front of a building which marks the centre, and which sometimes flanks a corner, when it is termed an angular pavilion.

Pedestal. The square support of a column, statue, etc.; and the base or lower part of an order of columns: it consists of a plinth for a base, the die, and a talon crowned for a cornice. When the height and width are equal, it is termed a square pedestal; one which supports two columns a double pedestal; and if it supports a row of columns without any break, it is a continued pedestal.

Pediment. A low triangular crowning, ornamented, in front of a building and over doors and windows. Pediments are sometimes made in the form of a segment; the space enclosed within the triangle is called the tympanum. Also the gable ends of classic buildings, where the horizontal cornice is carried across the front, forming a triangle with the end of the roof.

Pendent. A name given to an elongated boss, either molded or foliated, such as hang down from the intersection of groins, especially in fan tracery, or at the end of hammer beams. Sometimes long corbels, under the wall pieces, have been so called. The name has also been given to the large masses depending from enriched ceilings, in the later works of the Pointed style.

Pendent Posts. A name given to those timbers which hang down the side of a wall from the plate in hammer beam trusses, and which receive the hammer braces.

Pendentive. A name given to an arch which cuts off, as it were, the corners of a square building internally, so that the superstructure may become an octagon or a dome. In mediæval architecture these arches, when under a spire in the interior of a tower, are called Squinches.

Pendentive Bracketing, or Cove Bracketing. Springing from the rec-ngular walls of an apartment upward to the ceiling, and forming the horizon-l part of the ceiling into a circle or ellipse.

Pentastyle. Having five columns in front.

Pent-roof. A roof with a slope on one side only.

Perch. A measure used in measuring stone work, being 24¾ cu ft and 16½ ft, according to locality and custom.

Periptery. An edifice or temple surrounded by a peristyle.

Peristyle. A range of columns encircling an edifice, such as that which sur-unds the cylindrical drum under the cupola of St. Paul's. The columns of a eek peripteral temple form a peristyle also, the former being a circular, and e latter a quadrilateral peristyle.

Perpendicular Style. The third and last of the Pointed or Gothic styles; o called the Florid style.

Perspective Drawing. The art of making such a representation of an object on a plane surface as shall present precisely the same appearance that the ject itself would to the eye situated at a particular point.

Pews. A word of uncertain origin, signifying fixed seats in churches, com-sed of wood framing, mostly with ornamented ends. They seem to have come to general use early in the reign of Henry VI, and to have been rented and rell paid for" before the Reformation. Some bench ends are certainly of a corated character, and some have been considered to be of the Early English riod. They are sometimes of plain oak board, two and a half to three inches ick, chamfered, and with a necking and finial, generally called a poppy head; hers are plainly paneled with bold cappings; in others the panels are orna-nted with tracery or with the linen pattern, and sometimes with running iages. The divisions are filled in with thin chamfered boarding, sometimes ching to the floor, and sometimes only from the capping to the seat.

Picket. A narrow board, often pointed, used in making fences; a pale or ling.

Pier-glass. A mirror hanging between windows.

Piers. The solid parts of a wall between windows, and between voids gener-y. The term is also applied to masses of brick-work or masonry which are ulated to form supports to gates or to carry arches, posts, girders, etc.

Pilasters. Are flat square columns, attached to a wall, behind a column, or ng the side of a building, and projecting from the wall about a fourth or a th part of their breadth. The Greeks had a slightly different design for the pitals of pilasters, and made them the same width at top as at bottom, but the mans gave them the same capitals as the columns, and made them of di-nished width at the top, similar to the columns.

Pile. A large stake or trunk of a tree, driven into soft ground, as at the ttom of a river, or in made land, for the support of a building. (See p. 188.)

Pillar, or Pyller. A word generally used to express the round or polygonal rs, or those surrounded with clustered columns, which carry the main arches a building. Saxon and Early Norman pillars are generally stout cylindrical ifts built up of small stones. Sometimes, however, they are quite square, netimes with other squares breaking out of them (this is more common in nch and German work), sometimes with angular shafts, and sometimes they plain octagons. In Romanesque Norman work the pillar is sometimes iare, with two or more semicircular or half-columns attached. In the Early

English period the pillars become loftier and lighter, and in most important buildings are a series of clustered columns, frequently of marble, placed side by side, sometimes set at intervals round a circular centre, and sometimes almost touching each other. These shafts are often wholly detached from the central pillar, though grouped round it, in which case they are almost always of Purbeck or Bethersden marbles. In Decorated work the shafts on plan are very often placed round a square set anglewise, or a lozenge, the long way down the nave; the centre or core itself is often worked into hollows or other moldings, to show between the shafts, and to form part of the composition. In this and the latter part of the previous style there is generally a fillet on the outer part of the shaft, forming what has been called a keel molding. They are also often, as it were, tied together by bands formed of rings of stone and sometimes of metal. The small pillars at the jambs of doors and windows, and in arcades, and also those slender columns attached to pillars, or standing detached, are generally called shafts.

Pin. A cylindrical piece of wood, iron, or steel, used to hold two or more pieces together, by passing through a hole in each of them, as in a mortise and tenon joint, or a pin joint of a truss.

Pinnacle. An ornament originally forming the cap or crown of a buttress or small turret, but afterwards used on parapets at the corners of towers and in many other situations. It was a weight to counteract the thrust of the groining of roofs, particularly where there were flying buttresses; it stopped the tendency to slip of the stone copings of the gables, and counterpoised the thrust of spires; it formed the piers to steady the elegant perforated parapets of later periods; and in France, especially, served to counterbalance the weight of overhanging corbel tables, huge gargoyles, etc. In the Early English period the smaller buttresses frequently finished with gablets, and the more important with pinnacles supported with clustered shafts. At this period the pinnacles were often supported on these shafts alone, and were open below; and in larger work in this and the subsequent periods they frequently form niches and contain statues. In France, pinnacles, like spires, seem to have been in use earlier than in England. There are small pinnacles at the angles of the tower in the Abbey of Saintes. At Roullet there are pinnacles in a similar position, each composed of four small shafts, with caps and bases surmounted with small pyramidal spires. In all these examples the towers have semicircular headed windows.

PINNACLE.

Pitch of a Roof. The proportion obtained by dividing the height by the span; thus, we speak of its being one-half, one-third, one-fourth. When the length of the rafters is equal to the breadth of the building it is denominated Gothic.

Pitching-piece. A horizontal timber, with one of its ends wedged into the wall at the top of a flight of stairs, to support the upper end of the rough string.

Place. An open piece of ground surrounded by buildings, generally decorated with a statue, column, or other ornament.

Plan. A horizontal geometrical section of the walls of a building; or indications, on a horizontal plane, of the relative positions of the walls and partitions with the various openings, such as windows and doors, recesses and projections, chimneys and chimney-breasts, columns, pilasters, etc. This term is often incorrectly used in the sense of Design.

Planceer. Is sometimes used in the same sense as soffit, but is more correctly plied to the soffit of the corona in a cornice.

Plastering. A mixture of lime, hair, and sand, to cover lath-work between nbers or rough walling, used from the earliest times, and very common in man work. In the Middle Ages, too, it was used not only in private, but in blic constructions. On the inside face of old rubble walls it was not only used purposes of cleanliness, rough work holding dirt and dust, but as a ground distemper painting (tempera, or, as it is often improperly called, fresco), a ecies of ornament often used in the Middle Ages. At St. Albans Abbey, Engd, the Norman work is plastered, and covered with lines imitating the joints stone. The same thing is found in English Perpendicular work. On the outle of rubble walls, and often of wood framing, it was used as roughcast; when namented in patterns outside, it is called pargeting.

Plate. The piece of timber in a building which supports the end of the ters.

Plinth. The square block at the base of a column or pedestal. In a wall, the m plinth is applied to the projecting base or water table, generally at the level the first floor.

Plumb. Perpendicular; that is, standing according to a plumb line, as, the st of a house or wall is plumb.

Plumbing. The lead and iron pipes and other apparatus employed in conying water, and for toilet purposes in a building; originally the art of casting d working in lead.

Ply. Used to denote the number of thicknesses of roofing paper, as three ply, r ply, etc.

Podium. A continued pedestal; a projection from a wall, forming a kind of llery.

Polytriglyph. An intercolumniation in the Doric order of more than two glyphs.

Poppy Heads. Probably from the French *poupès:* the finials or other ornants which terminate the tops of bench ends, either to pews or lls. They are sometimes small human heads, sometimes richly rved images, knots of foliage, or finials, and sometimes fleurs-lis simply cut out of the thickness of the bench end and chamed.

POPPY HEAD

Porch. A covered erection forming a shelter to the entrance or of a large building. The earliest known are the long arcaded rches in front of the early Christian basilicas, called Narthex. later times they assume two forms—one, the projecting erection vering the entrance at the west front of cathedrals, and divided to three or more doorways, etc.; and the other, a kind of covered ambers open at the ends, and having small windows at the sides a protection from rain.

Portal. A name given to the deeply recessed and richly decorated entrance ors to the cathedrals in Continental Europe.

Portcullis. A strong-framed grating of oak, the lower points shod with iron, d sometimes entirely made of metal, hung so as to slide up and down in grooves th counterbalances, and intended to protect the gateways of castles, etc.

Portico. An open space before the door or other entrance to any building, nted with columns. A portico is distinguished as prostyle or *in antis* accord-

ing as it projects from or recedes within the building, and is further designated by the number of columns its front may consist of.

Post. Square timbers set on end. The term is especially applied to those which support the corners of a building, and are framed into bressummers or crossbeams under the walls.

Posticum. A portico behind a temple.

Presbytery. A word applied to various parts of large churches in a very ambiguous way. Some consider it to be the choir itself; others, what is now named the sacrarium. Traditionally, however, it seems to be applied to the vacant space between the back of the high altar and the entrance to the lady-chapel, as at Lincoln and Chichester; in other words, the back- or retro-choir.

Priming. The laying on of the first shade of color, in oil paint, and generally consisting mostly of oil, to protect and fill the wood.

Priory. A monastic establishment, generally in connection with an abbey, and presided over by a prior, who was a subordinate to the abbot, and held much the same relation to that dignitary as a dean does to a bishop.

Profile. The outline; the contour of a part, or the parts composing an order, as of a base, cornice, etc.; also, the perpendicular section. It is in the just proportion of their profiles that the chief beauties of the different orders of architecture depend. The ancients were most careful of the profiles of their moldings.

Proscenium. The front part of the stage of ancient theaters, on which the actors performed.

Prostyle. A portico in which the columns project from the building to which it is attached.

Protractor. A mathematical instrument for laying down and measuring angles on paper, used in drawing or plotting.

Pseudo-dipteral. False double-winged. When the inner row of columns of a dipteral arrangement is omitted and the space from the wall of the building to the columns is preserved, it is pseudo-dipteral.

Puddle. To settle loose dirt by turning on water, so as to render it firm and solid.

Pugging. A coarse kind of mortar laid on the boarding, between floor joists, to prevent the passage of sound; also called deafening.

Pulpit. A raised platform with enclosed front, whence sermons, homilies, etc., were delivered. Pulpits were probably derived in their modern form from the ambones in the early Christian church. There are many old pulpits of stone, though the majority are of wood. Those in the churches are generally hexagonal or octagonal; and some stand on stone bases, and others on slender wooden stems, like columns. The designs vary according to the periods in which they were erected, having paneling, tracery, cuspings, crockets, and other ornaments then in use. Some are extremely rich, and ornamented with color and gilding. A few also have fine canopies or sounding boards. Their usual place is in the nave, mostly on the north side, against the second pier from the chancel arch. Pulpits for addressing the people in the open air were common in the Medieval period, and stood near a road or cross. Thus, there was one at Spitalfields, and one at St. Paul's, London. External pulpits still remain at Magdalen College, Oxford, and at Shrewsbury, England.

Purlins. Those pieces of timbers which support the rafters to prevent them from sinking.

Putlog. Horizontal pieces for supporting the floor of a scaffold, one end being serted into putlog holes, left for that purpose in the masonry.

Putty in Plastering. Lump lime slacked with water to the consistency of eam, and then left to harden by evaporation till it becomes like soft putty. It then mixed with plaster of Paris, or sand, for the finishing coat.

Puzzolana. A grayish earth used for building under water.

Pyramid. A solid, having one of its sides, called a base, a plane figure, and ne other sides triangles, these points joining in one point at the top, called the ertex. Pyramids are called triangular, square, etc., according to the form of eir bases.

Pyx. In Roman Catholic churches, the box in which the host, or consecrated afer, is kept.

Quadrangle. A square or quadrangular court surrounded by buildings, as as often done formerly in monasteries, colleges, etc.

Quarry. A pane of glass cut in a diamond or lozenge form.

Quarry-face. Ashlar as it comes from the quarry, squared off for the joints nly, with split face. In distinction from Rock-face, in that the latter may be eather-worn, while Quarry-face should be fresh split. The terms are often sed indiscriminately.

Quatrefoil. Any small panel or perforation in the form of a four-leaved flower. ometimes used alone, sometimes in circles and over the aisle windows, but more requently in square panels. They are generally cusped, and the cusps are often eathered.

Queen Truss. A truss framed with two vertical tie-posts, in distinction from he king-post, which has but one. The upright ties are called Queen-posts.

Quirk Moldings. The convex part of Grecian moldings when they recede at he top, forming a reëntrant angle, with the surface which covers the moldings.

Quoins. Large squared stones at the angles of buildings, buttresses, etc., enerally used to stop the rubble or rough stone work, and that the angles may e true and stronger. Saxon quoin stones are said to have been composed of ne long and one short stone alternately. Early quoins are generally roughly xed; in later times they had a draught tooled by the chisel round the outside dges, and later still were worked fine from the saw.

Rafters. The joist to which the roof boarding is nailed. *Principal rafters* are the upper timbers in a truss, having the same inclination as the common rafters.

Rail. A piece of timber or metal extending from one post to another, as in fences, balustrades, staircases, etc. In framing and paneling, the horizontal pieces are called rails, and the perpendicular, *stiles.*

Raking. Moldings whose arrises are inclined to the horizon.

Ramp. A concavity on the upper side of hand railings formed over risers, made by a sudden rise of the steps above. Any concave bend or slope in the cap or upper member of any piece of ascending or descending workmanship.

Rampant. A term applied to an arch whose abutments spring from an inclined plane.

Random Work. A term used by stone-masons for stones fitted together at random without any attempt at laying them in courses. *Random Coursed Work* is a like term applied to work coursed in horizontal beds, but the stones are of any height, and fitted to one another.

Range Work. Ashlar laid in horizontal courses; same as coursed ashlar.

Rebate. A groove on the edges of a board.

Recess. A depth of some inches in the thickness of a wall, as a niche, etc.

Refectory. The hall of a monastery, convent, etc., where the religious took their chief meals together. It much resembled the great halls of mansions, castles, etc., except that there frequently was a sort of ambo, approached by steps, from which to read the Legenda Sanctorum, etc., during meals.

Reglet. A flat, narrow molding, used to separate from each other the parts or members of compartments and panels, to form frets, knots, etc.

Renaissance (a new birth). A name given to the revival of Roman architecture which sprang into existence in Italy as early as the beginning of the fifteenth century, and reached its zenith in that country at the close of the century. There are several divisions of this style as developed in different localities: viz.,

The Florentine Renaissance, of which the Pitti Palace, by Brunelleschi, is one of the best examples.

The Venetian Renaissance, characterized by its elegance and richness.

The Roman Renaissance, which originated in Rome, under the architects known as Bronte, Vignola, and Michael Angelo. Of this style the Farnese Palace, St. Peter's, and the modern Capitol at Rome are the best examples.

The French Renaissance, introduced into France in the latter part of the fifteenth century, by Italian architects, where it flourished until the middle of the seventeenth century. The Renaissance style was introduced into Germany about the middle of the sixteenth century, and into England about the same time by John of Padua, architect to Henry VIII. This style in England is generally known under the name of Elizabethan.

Rendering. In drawing, finishing a perspective drawing in ink or color, to bring out the spirit and effect of the design. The first coat of plaster on brick or stone work.

Reredos, Dorsal, or **Dossel.** The screen or other ornamental work at the back of an altar. In some large English cathedrals, as Winchester, Durham, St. Albans, etc., this is a mass of splendid tabernacle work, reaching nearly to the groining. In smaller churches there are sometimes ranges of arcades or panelings behind the altars; but, in general, the walls at the back and sides of them were of plain masonry, and adorned with hangings or paraments. In the large churches of Continental Europe the high altar usually stands under a sort of canopy or ciborium, and the sacrarium is hung round at the back and sides with curtains on movable rods.

Reticulated Work. That in which the courses are arranged in a form like the meshes of a net. The stones or bricks are square and placed lozenge-wise.

Return. The continuation of a molding, projection, etc., in an opposite direction.

Return Head. One that appears both on the face and edge of a work.

Reveal. The two vertical sides of an aperture, between the front of a wall and the window or door frame.

Rib. A molding or projecting piece upon the interior of a vault, or used to form tracery and the like. The earliest groining had no ribs. In early Norman times plain flat arches crossed each other, forming ogive ribs. These by degrees became narrower, had greater projection, and were chamfered. In later Norman work the ribs were often formed of a large roll placed upon the flat band, and then of two rolls side by side with a smaller roll or a fillet between them.

uch like the lower member. Sometimes they are enriched with zigzags and ther Norman decorations, and about this time bosses became of very general se. As styles progressed, the moldings were more undercut, richer, and more laborate, and had the dog-tooth or ball-flower or other characteristic ornament the hollows. In all instances the moldings are of similar contours to those f arches, etc., of the respective periods. Later, wooden roofs are often formed nto cants or polygonal barrel vaults, and in these the ribs are generally a cluster f rounds, and form square or stellar panels, with carved bosses or shields at the ntersections.

Ridge. The top of a roof which rises to an acute angle.

Ridge-pole. The highest horizontal timber in a roof, extending from top to op of the several pairs of rafters of the trusses, for supporting the heads of the ack rafters.

Rilievo, or Relief. The projection of an architectural ornament.

Rise. The distance through which anything rises, as the rise of a stair, or nclined plane.

Riser. The vertical board under the tread in stairs.

Rococo Style. A name given to that variety of the Renaissance which was in ogue during the seventeenth and the latter part of the sixteenth century.

Romanesque Style. The term Romanesque embraces all those styles of arhitecture which prevailed between the destruction of the Roman Empire and the eginning of Gothic architecture. In it are included the Early Roman Christian rchitecture, Byzantine, Mahometan, and the later Romanesque architecture roper, which was developed in Italy, France, England, and Germany. This ater Romanesque, which was quite different from the preceding, came into vogue luring the tenth century, and reached its height during the twelfth century, and n the thirteenth century gave way to the Pointed or Gothic style. In England, Romanesque architecture is known under the name of the Saxon, Norman, and Lombard styles, according to the different political periods.

Rood. A name applied to a crucifix, particularly to those which were placed n the rood-loft or chancel screens. These generally had not only the image of he crucified Saviour, but also those of St. John and the Virgin Mary standing ne on each side. Sometimes other saints and angels are by them, and the top of the screen is set with candlesticks or other decoration.

Rood-loft, Rood-screen, Rood-beam, Jube Gallery, etc. The arrangement to carry the crucifix or rood, and to screen off the chancel from the rest of the church during the breviary services, and as a place whence to read certain parts of those services. Sometimes the crucifix is carried simply on a strong transverse beam, with or without a low screen, with folding-doors below but forming no part of such support. In European churches the general construction of wooden screens is close paneling beneath, about 3 feet to 3 feet 6 inches high, on which stands screen work composed of slender turned balusters or regular wooden mullions, supporting tracery more or less rich, with cornices, cresting, etc., and often painted in brilliant colors and gilded. These not only enclose the chancels, but also chapels, chantries, and sometimes even tombs. In English mansions, and some private houses, the great halls were screened off by a low passage at the end opposite to the dais, over which was a gallery for the use of minstrels or spectators. These screens were sometimes close and sometimes glazed.

Rood-tower. A name given by some writers to the central tower, or that over the intersection of the nave and chancel with the transepts.

Roof. The covering or upper part of any building.

Roofing. The material put on a roof to make it water-tight.

Rose Window. A name given to a circular window with radiating tracery; called also wheel window.

Rostrum. An elevated platform from which a speaker addresses an audience.

Rotunda. A building which is round both within and without. A circular room under a dome in large buildings is also called the rotunda.

Roughcast. A sort of external plastering in which small sharp stones are mixed, and which, when wet, is forcibly thrown or cast from a trowel against the wall, to which it forms a coating of pleasing appearance. Roughcast work has been used in Europe for several centuries, where it was much used in timber houses, and when well executed the work is sound and durable. The mortar for roughcast work should always have cement mixed with it.

Rubble Work. Masonry of rough, undressed stones. When only the roughest irregularities are knocked off, it is called scabbled rubble, and when the stones in each course are rudely dressed to nearly a uniform height, ranged rubble.

Rudenture. The figure of a rope or staff, which is frequently used to fill up the flutings of columns, the convexity of which contrasts with the concavity of the flutings, and serves to strengthen the edges. Sometimes, instead of a convex shape, the flutings are filled with a flat surface; sometimes they are ornamentally carved, and sometimes on pilasters, etc. Rudentures are used in relief without flutings, as their use is to give greater solidity to the lower part of the shaft, and secure the edges. They are generally only used in columns which rise from the ground and are not to reach above one-third of the height of the shaft.

Rustic or **Rock Work.** A mode of building in imitation of nature. This term is applied to those courses of stone work the face of which is jagged or picked so as to present a rough surface. That work is also called rustic in which the horizontal and vertical channels are cut in the joinings of stones, so that when placed together an angular channel is formed at each joint. *Frosted rustic work* has the margins of the stones reduced to a plane parallel to the plane of the wall, the intermediate parts having an irregular surface. *Vermiculated rustic work* has these intermediate parts so worked as to have the appearance of having been eaten by worms. *Rustic chamfered work,* in which the face of the stones is smooth, and parallel to the face of the wall, and the angles beveled to an angle of one hundred and thirty-five degrees with the face so that two stones coming together on the wall, the beveling will form an internal right angle.

Sacristy. A small chamber attached to churches, where the chalices, vestments, books, etc., were kept by the officer called the sacristan. In the early Christian basilicas there were two semicircular recesses or apsides, one on each side of the altar. One of these served as a sacristy, and the other as the bibliotheca or library. Some have supposed the sacristy to have been the place where the vestments were kept, and the vestry that where the priests put them on; but we find from Durandus that the sacrarium was used for both these purposes. Sometimes the place where the altar stands enclosed by the rails has been called sacrarium.

Saddle Bars. Narrow horizontal iron bars passing from mullion to mullion, and often through the whole window, from side to side, to steady the stone work, and to form stays, to which the lead work is secured. When the bays of the windows are wide, the lead lights are further strengthened by upright bars passing through eyes forged on the saddle bars, and called stanchions. When

addle bars pass right through the mullions in one piece, and are secured to the ambs, they have sometimes been called stay bars.

Sagging. The bending of a body in the middle by its own weight, or the load upon it.

Salient. A projection.

Salon. A spacious and elegant apartment for the reception of company, or or state purposes, or for the reception of paintings, and usually extending hrough two stories of the house. It may be square, oblong, polygonal, or ircular.

Sanctuary. That part of a church where the altar is placed; also, the most acred or retired part of a temple. A place for divine worship; a church.

Sanctus Bell-cot, or **Turret.** A turret or enclosure to hold the small bell ounded at various parts of the service, particularly where the words "Sanctus," tc., are read. This differs but little from the common bell-cot, except that it is enerally on the top of the arch dividing the nave from the chancel. Sometimes, owever, the bell seems to have been placed in a cot outside the wall. In Engand sanctus bells have also been placed over the gables of porches. In Contiental Europe they run up into a sort of small slender spire, called *flèche* in rance, and *guglio* in Italy.

Saracenic Architecture. That Eastern style employed by the Saracens, and which distributed itself over the world with the religion of Mahomet. It is a nodification and combination of the various styles of the countries which they onquered.

Sarcophagus. A tomb or coffin made of stone, and intended to contain the ody.

Sash. The framework which holds the glass in a window.

Scabble. To dress off the rougher projections of stones for rubble masonry vith a stone axe or scabbling hammer.

Scagliola. An imitation of colored marbles in plaster work, made by a comination of gypsum, glue, isinglass, and coloring matter, and finished with a high olish, invented between 1600 and 1649.

Scantling. The dimensions of a piece of timber in breadth and thickness; lso, studding for a partition, when under five inches square.

Scarfing. The joining and bolting of two pieces of timber together transersely, so that the two appear as one.

Sconce. A fixed hanging or projecting candlestick.

Scotia. A concave molding, most commonly used in bases, which projects a leep shadow on itself, and is thereby a most effective molding under the eye, s in a base. It is like a reversed ovolo, or, rather, what the mold of an ovolo vould present.

Scratch Coat. The first coat of plaster, which is scratched to afford a bond or the second coat.

Screeds. Long narrow strips of plaster put on horizontally along a wall, and arefully faced out of wind, to serve as guides for plastering the wide intervals etween them.

Screen. Any construction subdividing one part of a building from another, s a choir, chantry, chapel, etc. The earliest screens are the low marble podia hutting off the chorus cantantium in the Roman basilicas, and the perforated ancelli enclosing the bema, altar, and seats of the bishops and presbyters. The chief screens in a church are those which enclose the choir or the place where

the breviary services are recited. In Continental Europe this is done not only by doors and screen work, but also, when these are of open work, by curtains, the laity having no part in these services. In England screens were of two kinds: one, of open wood-work, generally called rood-screens or jubes, and which the French call *grilles, clôtures du chœur;* the other, massive enclosures of stone work enriched with niches, tabernacles, canopies, pinnacles, statues, crestings, etc., as at Canterbury, York, Gloucester, and many other places.

Scribing. Fitting wood-work to an irregular surface.

Section. A drawing showing the internal heights of the various parts of a building. It supposes the building to be cut through entirely, so as to exhibit the walls, the heights of the internal doors and other apertures, the heights of the stories, thicknesses of the floors, etc. It is one of the species of drawings necessary to the exhibition of a Design.

Sedilia. Seats used by the celebrants during the pauses in the mass. They are generally three in number — for the priest, deacon, and sub-deacon — and are in England almost always a species of niches cut into the south wall of churches, separated by shafts or by a species of mullions, and crowned with canopies, pinnacles, and other enrichments more or less elaborate. The piscina and ambry sometimes are attached to them. In Continental Europe the sedilia are often movable seats; a single stone seat has rarely been found.

Set-off. The horizontal line shown where a wall is reduced in thickness, and, consequently, the part of the thicker portion appears projecting before the thinner. In plinths this is generally simply chamfered. In other parts of work the set-off is generally concealed by a projecting string. Where, as in parapets, the upper part projects before the lower, the break is generally hid by a corbel table. The portions of buttress caps which recede one behind another are also called set-offs.

Shaft. In Classical architecture that part of a column between the necking and the apophyge at the top of the base. In later times the term is applied to slender columns either standing alone or in connection with pillars, buttresses, jambs, vaulting, etc.

Shed Roof, or **Lean-to.** A roof with only one set of rafters, falling from a higher to a lower wall, like an aisle roof.

Shore. A piece of timber placed in an oblique direction to support a building or wall temporarily while it is being repaired or altered.

Shrine. A sort of ark or chest to hold relics. It is sometimes merely a small box, generally with a raised top like a roof; sometimes an actual model of churches; sometimes a large construction, like that of Edward the Confessor at Westminster, of St. Genevieve at Paris, etc. Many are covered with jewels in the richest way; that of San Carlo Borromeo, at Milan, is of beaten silver.

Sills. Are the timbers on the ground which support the posts and superstructure of a timber building. The term is most frequently applied to those pieces of timber or stone at the bottom of doors or windows.

Skewback. The inclined stone from which an arch springs.

Skirtings. The narrow boards which form a plinth around the margin of a floor, now generally called the base.

Sleeper. A piece of timber laid on the ground to receive floor joists.

Soffit. The lower horizontal face of anything as, for example, of an entablature resting on and lying open between the columns, or the under face of an arch where its thickness is seen.

ound Board. The covering of a pulpit to deflect the sound into a church.

all. Bad or broken brick; stone chips.

an. The distance between the supports of a beam, girder, arch, truss, etc.

andrel, or Spandril. The space between any arch or curved brace and the l label, beams, etc., over the same. The spandrels over doorways in Perpen-ilar works are generally richly decorated.

pecification. Architect's. The designation of the kind, quality, and ntity of work and material to go in a building, in conjunction with the working wings.

pire. A sharply pointed pyramid or large pinnacle, generally octagonal in gland, and forming a finish to the tops of towers. Timber spires are very nmon in England. Some are covered with lead in flat sheets, others with the ne metal in narrow strips laid diagonally. Very many are covered with ngles. In Continental Europe there are some elegant examples of spires of en timber work covered with lead.

Splayed. The jamb of a door, or anything else of which one side makes an lique angle with the other.

Springer. The stone from which an arch springs; in some cases this is a pital, or impost; in other cases the moldings continue down the pier. The west stone of the gable is sometimes called a springer.

Squinches. Small arches or corbeled set-offs running diagonally and, as it re, cutting off the corners of the interior of towers, to bring them from the uare to the octagon, etc., to carry the spire.

Squint. An oblique opening in the wall of a church; especially, in mediæval chitecture, an opening so placed as to afford a view of the high altar from the insept or aisles.

Staging. A structure of posts and boards for supporting workmen and aterial in building.

Stall. A fixed seat in the choir for the use of the clergy. In early Christian nes the thronus cathedra, or seat of the bishop, was in the center of the apsis bema behind the altar, and against the wall; those of the presbyters also were ainst the wall, branching off from side to side around the semicircle. In later nes the stalls occupied both sides of the choir, return seats being placed at the ds for the prior, dean, precentor, chancellor, or other officers. In general, in thedrals, each stall is surmounted by tabernacle work, and rich canopies, nerally of oak.

Stanchion. A word derived from the French *étançon*, a wooden post, applied the upright iron bars which pass through the eyes of the saddle bars or hori-ntal irons to steady the lead lights. The French call the latter *traverses*, the inchions *montants*, and the whole arrangement *armature*. Stanchions fre-iently finish with ornamental heads forged out of the iron.

Steeple. A general name for the whole arrangement of tower, belfry, spire, etc.

Stereobate. A basement, distinguished from the nearly equivalent term sty-bate by the absence of columns.

Stile. The upright piece in framing or paneling.

Stilted. Anything raised above its usual level. An arch is stilted when its ntre is raised above the line from which the arch appears to spring.

Stoop. A seat before the door; often a porch with a balustrade and seats on e sides.

Stoup. A basin for holy water at the entrance of Roman Catholic ch
Into which all who enter dip their fingers and cross themselves.

Straight Arch. A form of arch in which the intrados is straight, but w
joints radiating as in a common arch.

Strap. An iron plate for connecting two or more timbers, to which
screwed by bolts. It generally passes around one of the timbers.

Stretcher. A brick or block of masonry laid lengthwise of a wall.

String Board. A board placed next to the well-hole in wooden stairs, t
nating the ends of the steps. The string piece is the piece of board put
the treads and risers for a support, and forming the support of the stair.

String-course. A narrow, vertically faced and slightly projecting co
an elevation. If window-sills are made continuous, they form a string-c
but if this course is made thicker or deeper than ordinary window-sills, or c
a set-off in the wall, it becomes a blocking-course. Also, horizontal mold
running under windows, separating the walls from the plain part of the para
dividing towers into stories or stages, etc. Their section is much the sa
the labels of the respective periods; in fact, these last, after passing roun
windows, frequently run on horizontally and form strings. Like labels, the
often decorated with foliages, ball-flowers, etc.

Studs, or **Studding.** The small timbers used in partitions and outside wo
walls, to which the laths and boards are nailed.

Style. The term style in architecture has obtained a conventional mea
beyond its simpler one, which applies only to columns and columnar arra
ments. It is now used to signify the differences in the moldings, general
lines, ornaments, and other details which exist between the works of va
nations, and also those differences which are found to exist between the work
any nation at different times.

Stylobate. A basement to columns. Stylobate is synonymous with pedesta
but is applied to a continued and unbroken substructure or basement to column
while the latter term is confined to insulated supports. The Greek temples ge
erally had three or more steps all around the temple, the base of the colum
resting on the top step; this was the stylobate.

Subsellium. A name sometimes given to the seat in the stalls of churche
same as miserere.

Summer. A girder or main-beam of a floor; if supported on two-story pos
and open below, it is called a Brace-summer.

Surbase. A cornice or series of moldings on the top of the base of a pedesta
podium, etc.; a molding above the base.

Surface. To make plane and smooth.

Systyle. An intercolumniation to which two diameters are assigned.

Tabernacle. A species of niche or recess in which an image may be plac
They are generally highly ornamented and often surmounted with crocket
gables. The word tabernacle is also often used to denote the receptacle for rel
which was often made in the form of a small house or church.

Tabernacle Work. The rich ornamental tracery forming the canopy, e
to a tabernacle, is called tabernacle work; it is common in the stalls and scre
of cathedrals, and in them is generally open or pierced through.

Tail Trimmer. A trimmer next to the wall, into which the ends of joists a
fastened to avoid flues.

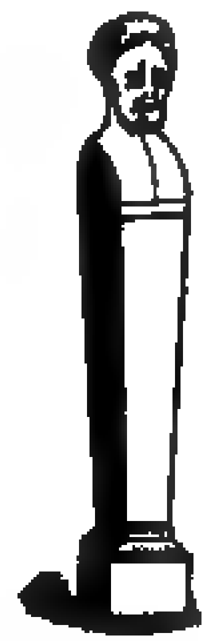
Tamp. To pound the earth down around a wall after it has been thrown in.

Tapestry. A kind of woven hangings of wool or silk, ornamented with figures, ıd used formerly to cover and adorn the walls of rooms. They were often of ɛ most costly materials and beautifully embroidered.

Temple. An edifice destined, in the earliest times, for the public exercise of ligious worship.

Templet, or **Template.** A mold used by masons for cutting or setting work. short piece of timber sometimes laid under a girder.

Terminal. Figures of which the upper parts only, or perhaps the head and oulders alone, are carved, the rest running into parallelopiped, and sometimes into a diminishing :destal, with feet indicated below, or even with- ıt them, are called terminal figures.



ANCIENT TERMINI

Terra-cotta. Baked clay of a fine quality. ıch used for bas-reliefs for adorning the friezes temples. In modern times employed for archi- ctural ornaments, statues, vases, etc.

Tessellated Pavements. Those formed of sseræ, or, as some write it, tesselle, or small ıbes from half an inch to an inch square, like ce, of pottery, stone, marble, enamel, etc.

Tetrastyle. A portico of four columns in front.

Tholobate. That on which a dome or cupola sts. This is a term not in general use, but it is not e less of useful application. What is generally rmed the attic above the peristyle and under the cupola of St. Paul's, London, ould be correctly designated the tholobate. A tholobate of a different descrip- ɔn, and one to which no other name can well be applied, is the circular sub- ructure to the cupola of the University College, London.

Throat. A channel or groove made on the under-side of a string-course, ping, etc., to prevent water from running inward toward the walls.

Tie. A timber, rod, chain, etc., binding two bodies together, which have a ndency to separate or diverge from each other. The *tie-beam* connects the ttom of a pair of principal rafters, and prevents them from bursting out the ıll.

Tiles. Flat pieces of clay burned in kilns, to cover roofs in place of slates or ıd. Also, flat pieces of burned clay, either plain or ornamented, glazed or ıglazed, used for floors, wainscoting, and about fireplaces, etc. Small square eces of marble are also called tile.

Tongue. The part of a board left projecting, to be inserted into a groove.

Tooth Ornament. One of the peculiar marks of the Early English period of othic architecture, generally inserted in the hollow moldings of doorways, ındows, etc.

Torso. A mutilated statue of which nothing remains but the trunk. Columns ıth twisted shafts have also this term. Of this kind there are several varieties.

Torus. A protuberance or swelling, a molding hose form is convex, and generally nearly approaches semicircle It is most frequently used in bases, and generally the lowest molding in a base.

TORUS

Tower. An elevated building originally designed for purposes of defense. Those buildings are of the remotest antiquity, and are, indeed, mentioned in the earliest Scriptures. In mediæval times they were generally attached to churches, to cemeteries, to castles, or used as bell-towers in public places of large cities. In churches, the towers of the Saxon period were generally square. Norman towers were also generally square. Many were entirely without buttresses; others had broad, flat, shallow projections which served for this purpose. The lower windows were very narrow, with extremely wide splays inside, probably intended to be defended by archers. The upper windows, like those of the preceding style, were generally separated into two lights, but by a shaft or short column, and not by a baluster. Early English towers were generally taller, and of more elegant proportions. They almost always had large projecting buttresses, and frequently stone staircases. The lower windows, as in the former style, were frequently mere arrow-slits; the upper were in couplets or triplets, and sometimes the tower top had an arcade all around. The spires were generally broach spires; but sometimes the tower tops finished with corbel courses and plain parapets, and (rarely) with pinnacles. There are a few Early English towers which break into the octagon from the square toward the top, and still fewer which finish with two gables. Both these methods of termination, however, are common in Continental Europe. At Vendôme, Chartres, and Senlis the towers have octagonal upper stages surrounded with pinnacles, from which elegant spires arise. In the North of Italy, and in Rome, they are generally tall square shafts in four to six stages, without buttresses, with couplets or triplets of semicircular windows in each stage, generally crenellated at top, and covered with a low pyramidal roof. The well-known leaning tower at Pisa is cylindrical, in five stories of arcaded colonnades. In Ireland there are in some of the churchyards very curious round towers.

Tracery. The ornamental filling in of the heads of windows, panels, circular windows, etc., which has given such characteristic beauty to the architecture of the fourteenth century. Like almost everything connected with mediæval architecture, this elegant and sometimes fairy-like decoration seems to have sprung from the smallest beginnings. The circular-headed window of the Normans gradually gave way to the narrow-pointed lancets of the Early English period, and, as less light was afforded by the latter system than by the former, it was necessary to have a greater number of windows; and it was found convenient to group them together in couplets, triplets, etc. When these couplets were assembled under one label, a sort of vacant space or spandrel was formed over the lancets and under the label. To relieve this, the first attempts were simply to perforate this flat spandrel, first by a simple lozenge-shaped or circular opening, and afterward by a quatrefoil. By piercing the whole of the vacant spaces in the window head, carrying moldings around the tracery, and adding cusps to it, the formation of tracery was complete, and its earliest result was the beautiful geometrical work such as is found at Westminster Abbey.

Transept. That portion of a church which passes transversely between the nave and choir at right angles, and so forms a cross on the plan.

Transom. The horizontal construction which divides a window into heights or stages. Transoms are sometimes simple pieces of mullions placed transversely as cross-bars, and in later times are richly decorated with cuspings, etc.

Traverse. To plane in a direction across the grain of the wood, as to traverse a floor by planing across the boards.

Tread. The horizontal part of a step of a stair.

Trefoil. A cusping the outline of which is derived from a three-leaved flower leaf, as the quatrefoil and cinque-foil are from those with four and five.

Trellis. Lattice-work of metal or wood for vines to run on.

Trestle. A movable frame or support for anything; when made of a cross ece with four legs it is called by carpenters a horse.

Triforium. The arcaded story between the lower range of piers and arches d the clere-story. The name has been supposed to be derived from *tres* and *es* — three doors, or openings — that being a frequent number of arches in ch bay.

Triglyph. The vertically channeled tablets of the Doric frieze are called glyphs, because of the three angular channels in them — two perfect and one vided — the two chamfered angles or hemiglyphs being reckoned as one. The uare sunk spaces between the triglyphs on a frieze are called metopes.

Trim. Of a door, sometimes used to denote the locks, knobs, and hinges.

Trimmer. The beam or floor joist into which a header is framed.

Trimmer Arch. An arch built in front of a fireplace, in the thickness of the or, between two trimmers. The bottom of the arch starting from the chimney d the top pressing against the header.

Tuck-pointing. Marking the joints of brickwork with a narrow parallel lge of fine putty.

Tudor Style. The architecture which prevailed in England during the reign the Tudors; its period is generally restricted to the end of the reign of Henry III.

Turret. A small tower, especially at the angles of larger buildings, sometimes erhanging and built on corbels, and sometimes rising from the ground.

Tuscan Order. The plainest of the five orders of Classic architecture.

Tympanum. The triangular recessed space enclosed by the cornice which ounds a pediment. The Greeks often placed sculptures representing subjects nnected with the purposes of the edifice in the tympana of temples, as at the arthenon and Ægina.

Under-croft. A vaulted chamber under ground.

Upset. To thicken, and shorten as by hammering a heated bar of iron on the id.

Vagina. The upper part of the shaft of a terminus, from which the bust or gure seems to rise.

Valley. The internal angle formed by two inclined sides of a roof.

Valley Rafters. Those which are disposed in the internal angle of a roof to orm the valleys.

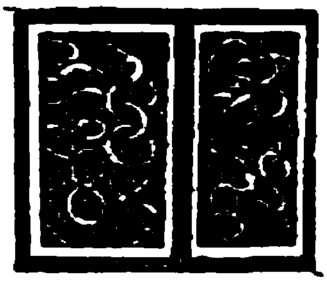
Vane. The weathercock on a steeple. In early times it seems to have been f various forms, as dragons, etc.; but in the Tudor period the favorite design as a beast or bird sitting on a slender pedestal, and carrying an upright rod, on hich a thin plate of metal is hung like a flag, ornamented in various ways.

Vault. An arched ceiling or roof. A vault is, indeed, a laterally conjoined eries of arches. The arch of a bridge is, strictly speaking, a vault. Intersecting aults are said to be groined. See *Groined Vaulting* for fuller description of aults.

Verge. The edge of the tiling, slate or shingles, projecting over the gable of a oof, that on the horizontal portion being called eaves.

Verge Board. Often corrupted into Barge Board; the board under the verge of gables, sometimes molded, and often very richly carved, perforated, and cusped, and frequently having pendants, and sometimes finials, at the apex.

Vermiculated. Stones, etc., worked so as to have the appearance of having been worked by worms.



VERMICULATED

Vestibule. An anti-hall, lobby, or porch.

Vestry. A room adjoining a church, where the vestments of the minister are kept and parish meetings held. In American Protestant churches, the Sunday-school room is often called the vestry.

Viaduct. A structure of considerable magnitude, and usually of masonry, for carrying a railway across a valley.

Vignette. A running ornament, representing, as its name imports, a little vine, with branches, leaves, and grapes. It is common in the Tudor period, and runs or roves in a large hollow or casement. It is also called Trayle.

Villa. A country house for the retreat of the rich.

Volute. The convolved or spiral ornament which forms the characteristic of the Ionic capital. Volute, scroll, helix, and cauliculus are used indifferently for the angular horns of the Corinthian capital.

Voussoir. One of the wedge-like stones which form an arch; the middle one is called the key-stone.

Wainscot. The wooden lining of walls, generally in panels.

Wall Plates. Pieces of timber which are placed on top of brick or stone walls so as to form the support to the roof of a building.

Warped. Twisted out of shape by seasoning.

Water Table. A slight projection of the lower masonry or brickwork on the outside of a wall a few feet above the ground as a protection against rain.

Weather Boarding. Boards lapped over each other to prevent rain, etc., from passing through.

Weathering. A slight fall on the top of cornices, window-sills, etc., to throw off the rain.

Wicket. A small door opening in a larger. They are common in mediæval doors, and were intended to admit single persons, and guard against sudden surprises.

Wind. A turn, a bend. A wall is *out of wind* when it is a perfectly flat surface.

Wing. A side building less than the main building.

Withes. The partition between two chimney flues in the same stack.

RCHITECTURAL TERMS AS DEFINED IN VARIOUS BUILDING LAWS

COMPILED BY THE AMERICAN ARCHITECT AND BUILDING NEWS, PAGE 150, VOL. XXXIII

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Terms Defined

[he following terms chance to be defined in sundry building codes, which are tioned in each case. The fact that other codes are not mentioned is not neces- ly a proof that the term is not also elsewhere in use as defined.]

djoining Owner. The owner of the premises adjoining those on which k is doing or to be done. [*District of Columbia.*]

lteration. Any change or addition except necessary repairs in, to, or upon building affecting an external, party, or partition wall, chimney, floor, or rway, and "to alter" means to make such change or addition. [*Boston and ver.*]

ppendages. Dormer-windows, cornices, moldings, bay-windows, towers, es, ventilators, etc. [*Chicago and Minneapolis.*]

reas. Sub-surface excavations adjacent to the building-line for lighting or tilation of cellars or basements. [*District of Columbia.*]

ttic Story. A story situated either in whole or in part in the roof. [*Denver District of Columbia.*]

ase. "The *base* of a brick wall" means the course immediately above the ndation wall. [*Cincinnati and Cleveland.*]

asement Story. One whose floor is 12″ or more below the sidewalk, and se height does not exceed 12′ in the clear; all such stories that exceed 12′ h shall be considered as first stories. [*Chicago and Louisville.*]

story whose floor is 12″ or more below the grade of sidewalk. [*Milwaukee.*]

story whose floor is 3′ or more below the sidewalk, and whose height does exceed 11′ in the clear; all such stories that exceed 11′ high shall be con- ered as first stories. [*Minneapolis.*]

story suitable for habitation, partially below the level of the adjoining street ground.* [*District of Columbia and Denver.*]

See **Cellar.**)

ay-window. A first-floor projection for a window other than a tower-pro- ion or show-window. [*District of Columbia.*]

ny projection for a window other than a show-window. [*Denver.*]

earing Walls. Those on which beams, trusses, or girders rest. [*New York San Francisco.*]

rick Building. A building the walls of which are built of brick, stone, iron, other substantial and incombustible materials. [*Boston, Denver, and Kansas y.*]

* And below the first floor of joists. [*District of Columbia.*]

Building. Any construction within the scope and purview of these r tions. [*District of Columbia.*]

Building Line. The line of demarcation between public and private [*District of Columbia.*]

Building Owner. The owner of premises on which work is doing or done. [*District of Columbia.*]

Business buildings shall embrace all buildings used principally for bu purposes, thus including, among others, hotels, theaters, and office-buil [*Chicago, Louisville, Milwaukee, and Minneapolis.*]

Cellar. Basement or lower story of any building, of which one-half or of the height from the floor to the ceiling is below the level of the street* joining.† [*Boston, Denver, and Kansas City.*]

Portion of building below first floor of joists, if partially or entirely below level of the adjoining parking, street, or ground, and not suitable for habita [*District of Columbia.*]

Cement-mortar. A proper proportion of cement and sand without the mixture of lime. [*Kansas City.*]

Division Wall. One that separates part of any building from another of the same building. [*Cincinnati and Cleveland.*]

Floor-bearing walls extending through buildings from front to rear, and s rating stores and tenements in buildings or blocks owned by the same pa *Minneapolis.*]

(See **Partition-wall.**)

Dwelling-house Class. All buildings except public buildings and build of the warehouse class. [*Cincinnati and Cleveland.*]

Shall not apply to buildings accommodating more than three families. [S *Francisco.*]

External Wall. Every outer wall or vertical enclosure of a building o than a party-wall. [*Boston, Cincinnati, Cleveland, Denver, District of Colu Kansas City, and Providence.*]

First Story. The story the floor of which is at or first above the level of th sidewalk or adjoining ground, the other stories to be numbered in regular s cession, counting upward. [*Denver and District of Columbia.*]

Footing Course. A projecting course or courses under base of foundati wall. [*Cincinnati and Cleveland.*]

Foundation. That portion of wall below level of street curb,‡ and, where wall is not on a street, that portion of wall below the level of the highest grou next to the wall. [*Boston, Kansas City, New York, and Providence.*]

Portion of exterior wall below surface of adjoining earth or pavement, a portion of partition or party wall below level of basement or cellar floor. [*Dis trict of Columbia and Denver.*]

Foundation, Basement, or Cellar Walls. That part of walls of building th is below the floor or joists, which are on or next above the grade line. [*Detroi*

Portion of the wall below the level of street curb, in front of the central line o building. [*San Francisco.*]

* Ground. [*Providence.*]

† And not suitable for habitation. [*Denver.*]

‡ "And serve as supports for piers, columns, girders, beams, or other wal [*New York.*]

ombustible Scantling Partition. One plastered on both sides upon iron or wire cloth, and filled in with brickwork 8″ high from floor, provided the ing is not over 80′ high. [*Chicago.*]

ombustible Roofing. Covered with not less than three (3) thicknesses ng-felt, and good coat of tar and gravel, or with tin, corrugated-iron, or other esisting material with standing-seam or lap-joint. [*Denver.*]

ngths. Walls are deemed to be divided into distinct *lengths* by return , and the length of every wall is measured from the center of one return wall e center of another, provided that such return walls are external or party -walls of the thickness herein required, and bonded into the walls so deemed divided. [*Cincinnati and Cleveland.*]

flammable Material. Dry goods, clothing, millinery, and the like in s, flyings or goods in factories, or other substance readily ignited by drops or flyings from electric lights. [*Minneapolis.*]

dging-house. A building in which persons are temporarily accommodated sleeping* apartments, and includes hotels. [*Boston and Kansas City.*]

ny building or portion thereof in which persons are lodged for hire for less a week at one time. [*District of Columbia and Providence.*]

ny building or portion thereof in which persons are lodged for hire tempo-y, and includes hotels. [*Denver.*]

ansard Roof. One formed with an upper and under set of rafters, the r set more inclined to the horizon than the lower set. [*Denver and District olumbia.*]

riel Window. A projection for a window above the first floor. [*Distria olumbia.*]

artition. An interior division constructed of iron, glass, wood, lath and ter, or other destructible natures. [*District of Columbia.*]

artition-wall. Any interior wall of masonry in a building. [*Boston, sas City, and Providence.*]

n interior wall of non-combustible material. [*District of Columbia.*]

ny interior division constructed of iron, glass, wood, lath and plaster, or combination of those materials. [*Denver.*]

ee Division Wall.)

arty-wall. Every wall used, or built, in order to be used, as a separation of or more buildings.† [*Boston, Cincinnati, Cleveland, Denver, Kansas City, Providence.*]

wall built upon dividing line between adjoining premises for their common [*District of Columbia.*]

arking. The space between the sidewalk and the building line. [*District olumbia.*]

arking Line. The line separating parking and sidewalk. [*District of umbia.*]

ublic Building. Every building used as church, chapel, or other place of lic worship; also every building used as a college, school, public hall, hospital, ater, public concert-room, public ball-room, public lecture-room, or for any lic assemblage. [*Boston, Chicago, Cincinnati, Cleveland, Denver Kansas y, and Minneapolis.*]

* Staying apartments. [*Kansas City.*]

† To be used jointly by separate buildings. [*Cincinnati and Cleveland.*]

Such buildings as shall be owned and occupied for public purposes by the State, the United States, the corporation of the City of Brooklyn, or other public schools within said city. [*Brooklyn.*]

Public Hall. Every theater, opera-house, hall, church, school, or other building intended to be used for public assemblage. [*Milwaukee and Louisville.*]

Return Wall. No wall subdividing any building shall be deemed a return wall, as before mentioned, unless it is two-thirds the height of the external party-walls. [*Cincinnati and Cleveland.*]

Shed. A skeleton structure for storage or shelter. [*District of Columbia.*]

Open structure, enclosed only on one side and end, and erected on the ground. [*San Francisco.*]

Open or closed board structure. [*Denver.*]

Show-window. A store-window in which goods are displayed for sale or advertisement. [*District of Columbia and Denver.*]

Square thereof. The square or level of the walls before commencing the pitch for roof. [*District of Columbia.*]

Standard Depth for Foundations. For brick and stone buildings, ... below curb line. [*San Francisco.*]

Standard Depth of Cellars. 16′, measured down from sidewalk grade at property line. [*Memphis.*]

Standard Iron Door. Made of No. 12 plate-iron, frame or continuous 2″ x 2″ x 3/8″ angle-iron, firmly riveted. Two panel doors, to have proper cross-bars, one panel on either side, fastened together with hooks or proper bolts at top and bottom, and with not less than two lever-bars. All doors hung on iron frames of 3/8″ x 4″ iron, securely bolted together through wall, swung on ... hinges, fitting close to frame all around; sill between doors, iron, brick, or stone, to rise not less than two (2) inches above floor on each side of opening. Lintel over door, brick, iron, or stone. Floors of basement, when doors are to same, stone or cement, in no case wood. [*Denver.*]

Standard Skylight. Constructed of wrought-iron frames, with hammered or desk-light glass not less than 1/2″ thick; not larger than 10′ by 12′, except by special permission of the Inspector. [*Denver.*]

Storehouse. (See **Warehouse Class.**)

Street. All streets, avenues, and public alleys. [*Minneapolis.*]

Tenement-house. A building which, or any portion of which, is to be occupied, or is occupied, as a dwelling by more than three* families living independently of one another, and doing their cooking upon the premises. [*Boston, Denver, and Kansas City.*]

Or by more than two families† above the second floor, so living and cooking. [*Boston and Kansas City.*]

Building which shall contain more than two rooms in front on each floor, or which shall be built with a passage or arched way between distinct parts of the same building, or which building shall be intended for the separate accommodation of different families or occupants. [*Charleston.*]

Theater. Public hall containing movable scenery or fixed scenery which is not made of metal, plaster, or other incombustible material. [*Chicago, Louisville, and Milwaukee.*]

* *Two* instead of three. [*District of Columbia and Minneapolis.*]

† Upon one floor, but having a common right in the halls, stairways, yards, etc. [*Providence.*]

hickness of a Wall. The minimum thickness of such wall.* [*Boston, innati, Cleveland, Kansas City, Milwaukee, and Providence.*]

inned Covered Fire-door. Wood doors or shutters, double thickness of d, cross or diagonal construction, covered on both sides and all edges with t-tin, joints securely clinched and nailed. [*Denver.*]

ower Projection. A projection designed for an ornamental door-entrance, ornamental windows, or for buttresses. [*District of Columbia.*]

ault. An underground construction beneath parking or sidewalk. [*District olumbia.*]

eneered Building. Frame structure, the walls covered above the sill by a rall of brick, instead of clapboards. [*Common understanding in Chicago, vaukee, and Minneapolis, but not defined by law.*]

arehouse Class. Buildings used for the storage of merchandise, manufac-s in which machinery is operated, breweries, and distilleries. [*Cincinnati St. Louis.*]

idth of buildings shall be computed by the way the beams are placed; the thwise of the beams shall be considered and taken to be the widthwise of the ding. [*New York and San Francisco.*]

holesale store, or storehouse, shall embrace all buildings used (or intended e used) exclusively for purpose of mercantile business or storage of goods. *cago, Louisville, and Milwaukee.*]

ooden Building. A wooden or frame† building. [*Boston, Kansas City, Minneapolis.*]

ny building of which an external or party wall is constructed in whole or in of wood. [*Denver and District of Columbia.*]

aving more wood on the outside than that required for the door and window es, doors, shutters, sash porticos, and wooden steps, and all frame buildings heds, although the sides and ends are proposed to be covered with corrugated or other metal, shall be deemed a wooden building under this law. [*Charles-and Nashville.*]

* As applied to solid walls. [*Minneapolis and Providence.*]
† Or veneered. [*Minneapolis.*]

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By

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